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MULTI-LAYER PENSTOCKS AND HIGH PRESSURE WYES

Ewald Schmitz[1]
(Proc. Paper 1344)

ABSTRACT

Multi-layer penstock design and fabrication with field welding procedures is described with illustrations of installation in service. This construction results in field weldability without necessity for stress relieving, and provides lighter, safer, and economical penstocks. Construction features of several high pressure different wye designs are illustrated.

Description of Multi-Layer

Multi-layer penstocks are an adaptation of the multi-layer principle originated by the A. O. Smith Corporation of Milwaukee in finding a way for safer and more economical construction of high pressure vessels where the ratio of diameter to wall thickness was ten to one or less.[2] As the name implies, multi-layer construction involves the use of several layers of steel, each progressively wrapped and tightened around a cylinder by mechanical means and then welded together at the edges. The inner shell is the innermost band of a layer pipe and is made up in the same manner as any solid wall pipe. The thickness of the inner shell is from 3/8" to 1/2". The outer layers are approximately 1/4" in thickness. All layer longitudinal welds are staggered relative to each other and are ground flush prior to adding the subsequent layers. A cross section through a multi-layer pipe is shown in Fig. 1. The ends of the multi-layer shell sections are scarfed for welding to adjoining shell sections in order to make a pipe of desired length.

Note: Discussion open until January 1, 1958. Paper 1344 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO 4, August, 1957.

1. Chf. Engr., Specialty Products, Process Equipment Div., A. O. Smith Corp., Milwaukee, Wis.

2. "Multi-Layer Construction of Thick Wall Pressure Vessels" by T. M. Jasper and C. M. Scudder, Transactions American Institute of Chemical Engineers, 1941.

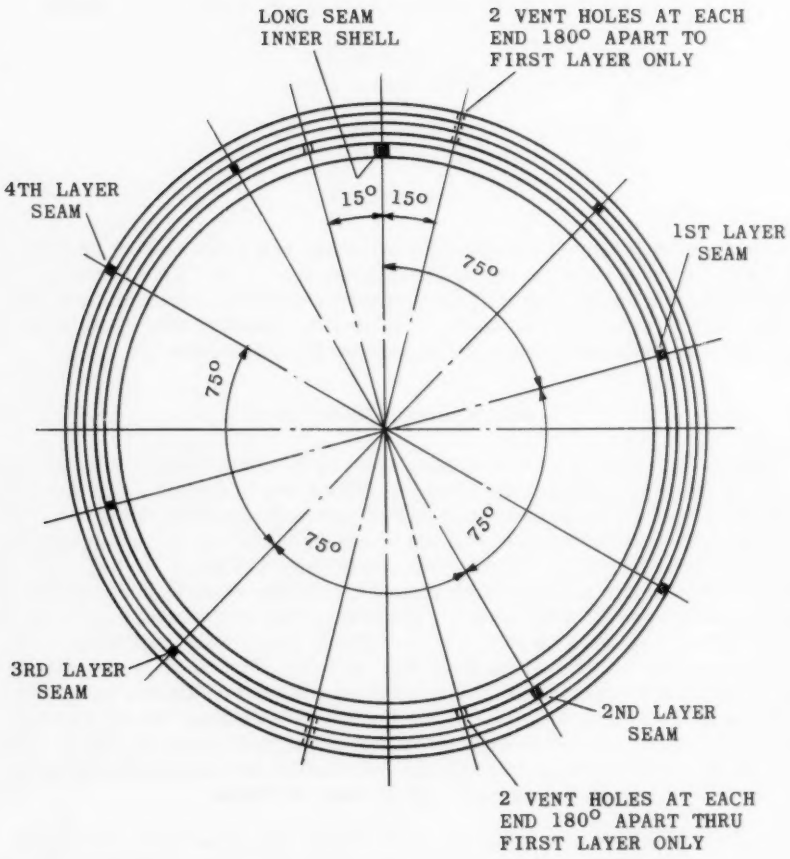


FIG. 1 TYPICAL SECTION THRU MULTI-LAYER PIPE

Characteristics of Multi-Layer Construction

The basic advantage of multi-layer construction is that it permits the use of thin plates for the resolution of high head penstock problems. The well-known advantages of thin plates over thick plates are:

1. Thorough visual inspection of plates after rolling at the steel mill. More area of plate can be inspected per ton of steel than when using equivalent single plate of same total thickness.
2. Higher yield and tensile strengths than those of thick plates of the same chemistry.
3. Certainty that ultimate shell strength will approach that of the steel plate. This permits a known factor of safety which avoids the necessity of using excess material to compensate for unknown values.
4. Negligible residual welding stresses after welding, which makes stress relieving unnecessary.
5. Less susceptibility to brittle fracture. Transition temperatures are inherently lower with thin materials.

The combination of the wrapping load imposed during fabrication of the multi-layer shell by the wrapping machine and the weld shrinkage of the longitudinal layer seams provides desirable stress distribution for working pressure anticipated in operation of the penstock. The multi-layer construction provides flexibility and the welding requirements are only those of thin wall shells. In welding the girth seams, the layers immediately above the top of the weld, as it is being built-up, do not conform to the shrinkage of the weld layer immediately below. This has the effect of having the weld shrinkage longitudinally affect only one layer at a time except at the longitudinal seam where a maximum of two layers are affected, and the shrinkage stress is only a fraction of that developed in solid wall vessels. Weld shrinkage in the circumferential direction induces desirable compressive stresses in the weld metal similar to those which are inherent in the multi-layer shell construction. Stress relieving of multi-layer vessels, therefore, is not required or desired. This applies both in the shop and the field.

Further proof of this was exhibited by fabricating two identical layer vessels. The layers came from steel plates that were split in half and each half was used in each vessel in identical positions. One vessel was stress relieved at 1150°F and the other was not. The physical properties of the steel in each vessel were determined by test coupons cut from each plate. Both vessels were tested to destruction, and the vessel which was not stress relieved developed a slightly higher burst test pressure than the other. Both vessels failed in the shell section and provided a ductile fracture without any shattering or fragmentation and developed their full calculated strength. Stress relieving is not recommended and has not been used on over 8,000 vessels fabricated of this construction to date. Many of these vessels operate at pressures up to 22,500 psi and with wall thicknesses up to 14".

The worthiness and the inherent safety features of this construction have been proven throughout almost the entire industry and by some unusual and severe service applications. The U.S. Navy has a non-shatterable air flask specification which requires that an air flask, when subjected to a gunfire test at a distance of 50 feet when fired with a 1.1 inch shell while the flask is under 3,000 psi air pressure, must not fragment. On a recent test of a multi-layer vessel designed for this purpose, the 1.1 inch shell did not penetrate the

wall of the vessel. It was agreed to fire at it with a 40 mm. shell (approximately 1-9/16" diameter). The 40 mm. shell penetrated the wall and did not fragment the flask.

A phase of a very interesting and revealing test program has recently been concluded. It was conducted on a full scale multi-layer vessel, which had been in daily use for 13 years from 1941 to 1954 serving as an accumulator for a hydraulic press and has had during that time approximately 4-1/2 million pressure cycles varying from one-half to the full working pressure of 4,500 psi. It was decided to take this vessel out of service and to subject it to an accelerated fatigue test. By means of an elaborate testing set-up, additional 5 million pressure cycles have been applied at a rate of approximately 25,000 cycles per day with a pressure variation from 4,000 to 5,500 psi.

Another interesting feature of this construction is that all layers having welded girth joints are provided with vent holes as indicated in Fig. 1. The vent holes constitute an automatic safety device and warning in the event of pitting corrosion penetration of the inner shell without causing damage to the penstock load carrying layers.

Because of the relatively light thickness of the inner shell and the thin layers around it, a readily weldable carbon steel can be used having a higher yield and ultimate strength than the thicker plate which would be required in single wall construction. Multi-layer means a penstock of less wall thickness with substantial savings in cost of material. The saving in weight is an appreciable item as indicated by the following comparisons:

Penstocks In Brazil	Single Wall	Multi-Layer	Tons Saved	% Saved
Lages C	530.5 Tons	479.3 Tons	51.2	10.4
Serra #4, 6 & 8	6,000 Tons	5,328 Tons	672	11.2

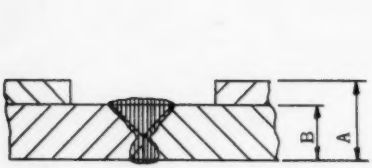
Another advantage of multi-layer construction stems from the well-known fact that in a penstock the longitudinal stresses are much less than the circumferential stresses. It is, therefore, sufficient to weld the girth seams to only about sixty per cent of the total thickness. Because of the flexibility of multi-layer construction, it is possible to take advantage of this design feature without getting the notch effect which would occur in partially welded single wall seams. This results in appreciable savings in time and money during the erection of the penstock. Fig. 2 shows a close-up view of two of the three penstocks built in this manner for the Sao Paulo Light & Power Company in Brazil. The various thicknesses of layers welded and not welded are given in Table 1.

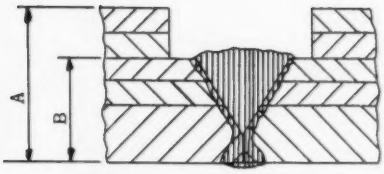
Test Results

The feasibility of multi-layer penstocks has been substantiated by testing to destruction a multi-layer penstock pipe section 36" I.D. of 1" nominal wall thickness and 12'-0" long, closed at the ends with welded-on test heads. Referring to Fig. 3 which shows an outline of the test vessel, L-1 to L-4 inclusive indicate the relative position of the longitudinal weld seams of the inner shells; C_1 to C_5 inclusive are the girth seams, a section through one is shown

MULTI-LAYER PENSTOCKS FOR SAO PAULO L & P CO. SERRA NO. 6 IS AT LEFT AND NO. 4 IS SECOND FROM LEFT. NOTE UN-WELDED GAPS AT CIRCUMFERENTIAL JOINTS. THE PENSTOCKS AT RIGHT ARE OF OLDER RIVETED DESIGN.

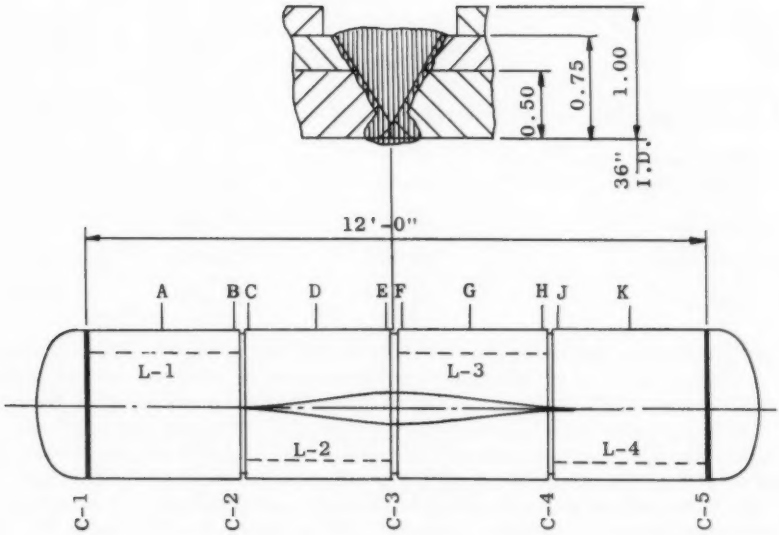
FIG. 2





A DIM.	B DIM.	% WELD	A DIM.	B DIM.	% WELD
0.625	0.437	70	1.062	0.781	73
0.656	0.437	66	1.093	0.718	65
0.687	0.437	63	1.125	0.687	61
0.718	0.468	65	1.156	0.718	62
0.750	0.500	66	1.187	0.750	63
0.781	0.500	64	1.218	0.750	61
0.875	0.687	78	1.250	0.750	60
0.906	0.687	75	1.281	0.781	61
0.937	0.687	73	1.312	0.937	71
0.968	0.750	77	1.343	0.968	72
1.000	0.750	75	1.375	1.000	72
1.031	0.750	72	1.406	0.968	68

TABLE NO. 1



CIRC. STRETCH IN INCHES AFTER 3000 PSI (LINE 1), 3615 PSI (LINE 2) & 4260 PSI (LINE 3)

	A	B	C	D	E	F	G	H	J	K
1	0.47	0.50	0.62	0.56	0.59	0.47	0.31	0.34	0.25	0.22
2	1.72	2.03	2.12	1.97	2.16	1.91	1.53	1.69	1.56	1.22
3	2.38	3.47	3.72	4.81	5.50	4.03	3.09	2.44	1.91	1.50

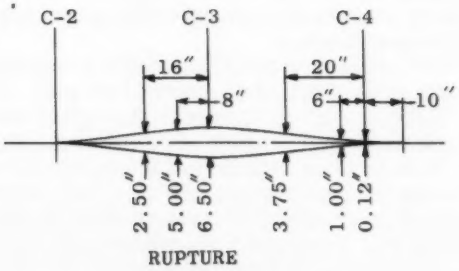


FIG. 3 BURST TEST DATA

shown in larger detail. Each of the four shell courses is 36" long. The physical properties of the inner shells and layers are tabulated below:

	Physicals From Plate Edge		
PART	Y. P. PSI DROP OF BEAM	ULT. T.S. PSI	% ELONG. IN 2"
Inner Shell of L-3 and L-4 Section	55,900	87,700	36
Inner Shell of L-1 and L-2 Section	51,800	79,900	39
1st Layer Half	57,400	78,900	33
2nd Layer Half	53,600	79,600	34
2nd Layer Half	59,100	83,100	34.5

The longitudinal seams of the inner shell only are x-rayed before application of the layers. The girth seams have been machine scarfed, except seam C-2 has been scarfed by flame cutting to simulate field scarfing when necessary. In accordance with normal practice, the pipe, designed for 1,210 psi pressure, was given a preliminary hydrostatic test of 1,950 psi, to produce a stress of 80% of the minimum specified yield strength of 45,000 psi. Tape readings taken before and after the preliminary test at points A to K inclusive showed an increase in circumference of 1/32" at points D and K and 1/64" at point G. Fig. 4 shows the pressure-water column curve, plotted from readings taken of the water input into the vessel at each 200 psi increment of pressure up to 3,000 psi. The yield pressure at 0.2% offset was determined from the curve as 2,760 psi. The corresponding hoop stress derived from equation (1) was,

$$S = \frac{2760\ (36 \not/ 1.2)}{2 \times 1} = 51{,}300 \text{ psi.}$$

The average theoretical yield strength of the material was 55,560 psi as determined by the drop of the beam. The vessel yield strength comparative to the drop of beam method is equal to a volumetric offset of 0.4% and was reached at a pressure of 2,890 psi which produced a stress of 53,750 psi. This stress is 96.5% of the yield strength of the actual material, or 119% of the minimum in the specification.

The pipe burst at a pressure of 4,260 psi with a corresponding stress, calculated on the basis of the original diameter of the pipe, of 79,200 psi which is in excess of the 75,000 psi minimum tensile strength of the material specification. Fig. 5 shows a view of the pipe after bursting. Examination of the fracture showed freedom from defects, and it was characteristic of a normal, clean break. The extent of the rupture and the actual circumferential stretch determined from measurements at the outside of the pipe at different pressures is tabulated in Fig. 3 and indicates considerable ductility before rupture.

The tensile strength of the L-2 section averages 80,375 psi. Therefore, the stress of 79,200 psi developed in the pipe at burst is equal to 98.5% of the ultimate strength of the actual material in that section, or 107% of the minimum in the specification.

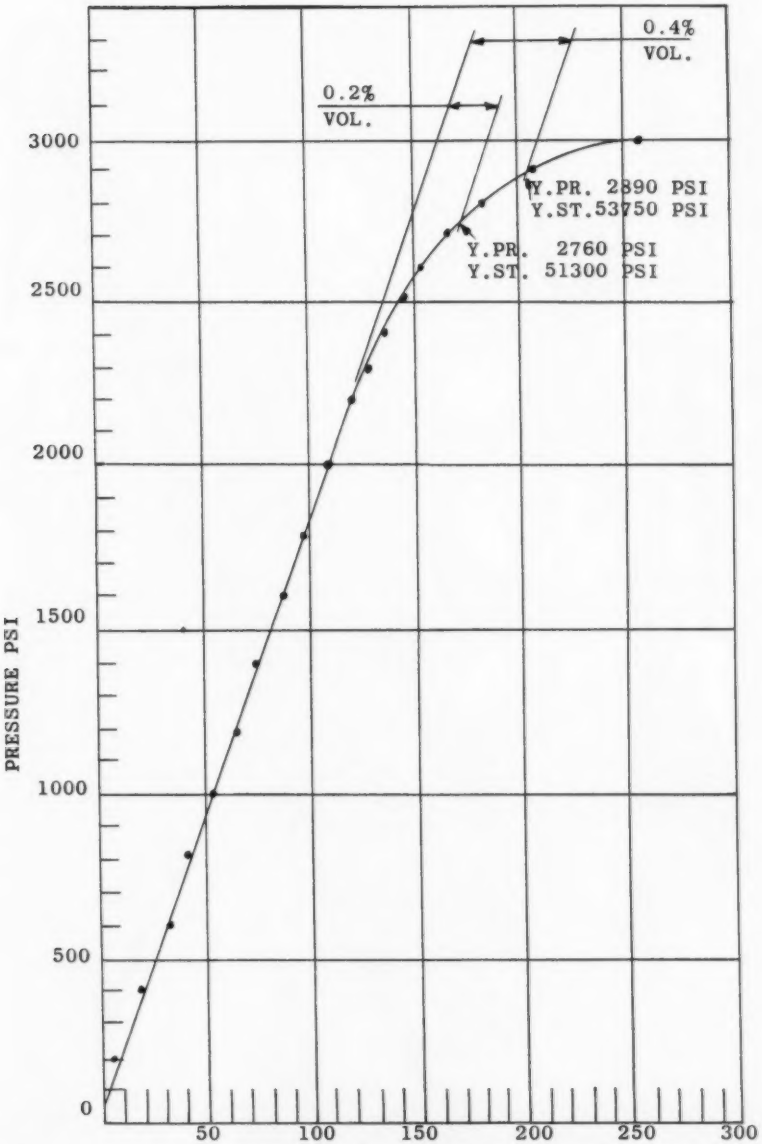


FIG. 4 PRESSURE-WATER COLUMN CURVE

FIG. 5 RUPTURED VESSEL AFTER 4260 PSI BURSTING PRESSURE

Plate Material

Two grades of material are at present available for multi-layer construction of penstock pipe. One of these is a medium high tensile carbon steel made by the open hearth process. It is a modification of ASTM-A212 Grade B specification having a minimum tensile strength of 75,000 psi, a minimum yield strength of 45,000 psi, and an elongation in 2" of 28%. The chemical composition of the steel is:

Carbon	0.20% to 0.30%
Manganese	0.90% to 1.30%
Silicon	0.10% max.
Phosphorus	0.045% max.
Sulphur	0.050% max.

The temperature transition curves determined from standard Izod tests for the layer steel and the weld metal as welded with A. O. Smith Corporation SW-76 (E-7020) electrodes and for the heat affected zone are shown in Fig. 6. The transition temperature is zero degrees Fahrenheit.

The other material is a low alloy high yield point material and is a modification of ASTM-A225 Grade B specification and has a minimum tensile strength of 80,000 psi and minimum yield strength of 60,000 psi, and a minimum elongation in 2" of 20%. The chemical composition of the steel is:

Carbon	0.20% max.
Manganese	1.55% max.
Phosphorus	0.04% max.
Sulphur	0.05% max.
Silicon	0.15% to 0.30%
Vanadium	0.08% to 0.14%

The temperature transition curve for this material is shown in Fig. 7, and is based on tests made on 2/3 of standard size Charpy keyhole notched bars. It is noted that the impact properties are constant at 23 ft. lbs. down to minus 40° F.

Both steels are readily weldable under shop as well as under field conditions.

Design

The wall thickness of a high head multi-layer penstock is satisfactorily determined on the basis of a fibre stress of 50% of the minimum specified yield strength of the steel with a joint efficiency of 100%. The latter value is used because of the staggering of the longitudinal joints of the layers. Therefore, the formula for thin shells in Section VIII of the ASME Code for Unfired Pressure Vessels may be written

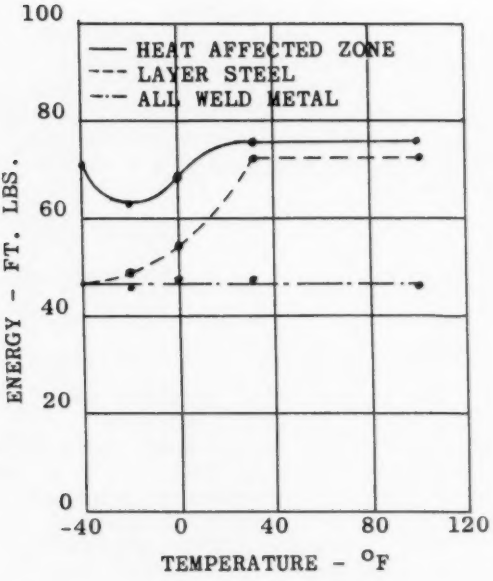


FIG. 6 IZOD IMPACT TESTS - TEMPERATURE TRANSITION CURVES OF CARBON STEEL LAYERS.

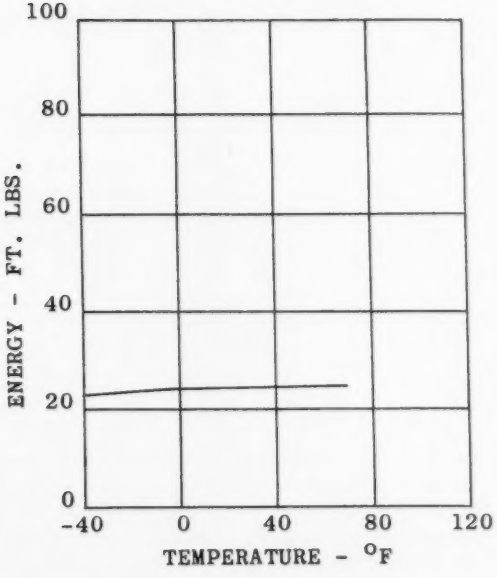


FIG. 7 2/3 SIZE CHARPY KEYHOLE IMPACT TESTS - TEMPERATURE TRANSITION CURVE OF LOW ALLOY LAYER MATERIAL.

$$t = \frac{PD}{2S - 1.2P} \quad (1)$$

where t is the wall thickness in inches, P is the design pressure in psi which includes allowance for water hammer, D is the inside diameter in inches, and S is the allowable hoop stress in psi. Allowance for corrosion or erosion may be added to the calculated thickness if so desired.

Substituting the hydrostatic head for P, equation (1) becomes

$$t = \frac{0.433\ HD}{2S - 0.52H} \quad (2)$$

in which H is the head in feet.

Multi-layer construction usually begins with a total wall thickness of 5/8" which can be increased in increments of 1/32" or 1/16" to satisfy increasing pressure requirements. Plain penstock pipes of a thickness less than 5/8" are of single wall construction and are not stress relieved.

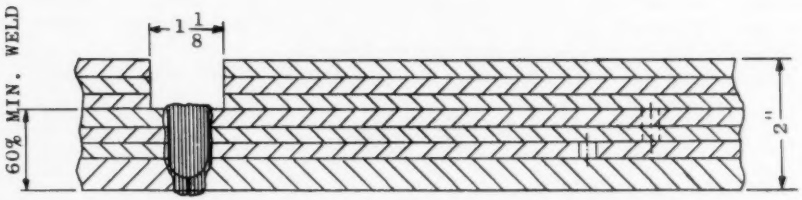
Expansion joints, taper pieces, wyes and all other special pipes such as pipe courses with thrust rings for bends that are to be made of single plate are completed and stress relieved in the manufacturer's plant. The wall thickness is determined on allowable stresses based on the physical properties of the steel used in the stress relieved condition.

In order to avoid the necessity of welding and stress relieving thick solid wall portions of the penstock in the field, the design is worked out so that any field joint exceeding 5/8" in thickness will be between two pipes, one of which will be of multi-layer construction. This is simply accomplished by shop welding multi-layer stub ends to the pipe of solid wall construction after the latter has been stress relieved. These stub ends are usually 2-1/2 feet long but can be made longer if so desired. Conditions of this type occur, for instance, when a solid wall expansion joint is to be welded to the down-stream end of a solid wall portion of a bend. The bend will then be provided with a multi-layer stub. The girth weld seam between a multi-layer and a solid wall pipe is always welded to the full wall thickness so that a notch is not left on the solid wall pipe. Typical details of the four types of girth seams used in shop and field practice are shown in Fig. 8.

End flanges for bolted connections, if they are made of forged steel, are usually of the welding neck type conforming to A.S.T.M. Specification A-105 Grade II except modified as follows:

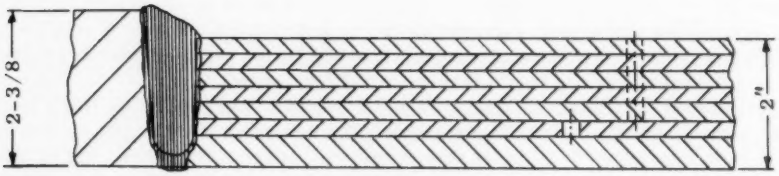
Carbon	0.30% max.
Manganese	1.25% max.
Phosphorus	0.045% max.
Sulphur	0.05% max.
Silicon	0.30% max.
Chromium	0.25% max.
Nickel	0.25% max.
Cobalt	0.20% max.

The physical properties of the steel as determined from test bars taken from the forgings after all heat treatments are completed are:



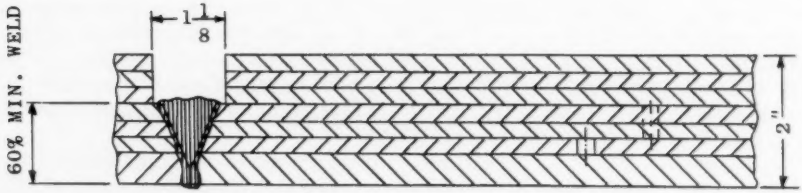
SHOP WELD MULTI-LAYER TO MULTI-LAYER

(a)



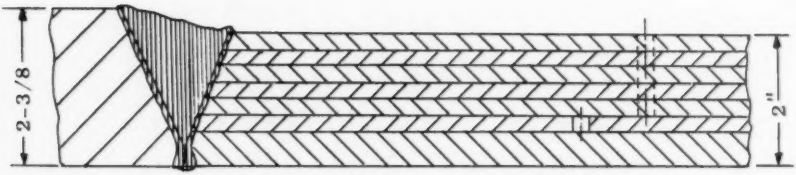
SHOP WELD SOLID WALL TO MULTI-LAYER

(b)



FIELD WELD MULTI-LAYER TO MULTI-LAYER

(c)



FIELD WELD SOLID WALL TO MULTI-LAYER

(d)

FIG. 8 TYPICAL MULTI-LAYER GIRTH SEAMS

Tensile Strength, psi . .	65,000 to 78,000
Yield Strength, psi . . .	33,000 to 45,000
Elong. in 2"	22%
Reduction in Area . . .	35%

The temperature impact curves representing a comparison of average test results between Charpy keyhole notch, Charpy V-notch and standard Izod values are shown in Fig. 9. The ASME Code uses either the Charpy keyhole notch specimen or the Izod notch specimen for service temperatures below minus 20° F. It is noted that either of these tests are above 15 ft. lbs. at minus 40° F.

In a high head penstock, the pipe diameters and wall thicknesses are usually such that 120 degree saddle piers for intermediate supports between anchor blocks can be used. The location and quantity of piers is dependent upon the ground condition, but general practice is to place one saddle per length of pipe. On previous installations saddle plates have been used, made from 3/8" thick steel plates suitably curved and provided with welded-on segments at both ends, and lubricating pipes. The saddle plates were concreted in at the piers. A suitable mineral lubricant is used to reduce saddle friction. Fig. 10 shows a view of the saddle piers for "Lages C" penstock.

Ring type supports for long spans and large pipe diameters are also feasible in connection with multi-layer penstocks. Two avenues are open to choose from in order to obtain the most satisfactory installation depending upon the evaluation of all factors governing the design. One way would be to make that portion of the pipe, to which the ring girder support is attached, of solid wall construction. The entire assembly could be completely shop fabricated, stress relieved and machined to assure accurate alignment of the bearing plates. Multi-layer stub ends could be shop welded to each end so that field welding will be remote from the support. The other and, no doubt, more economical way would be to apply an over-wrap layer of required length and thickness to the multi-layer pipe and weld the rings and column supports to the over-wrap. The stress analysis in either case may be made in accordance with the theory of thin cylindrical shells,[3] and as shown in actual examples by Bier.[4]

In respect to bending stresses due to long spans, the behavior of multi-layer pipe at mid-span is not different from that of a single wall shell of equal total thickness. A 36" I.D. multi-layer vessel 725 feet long is being operated as an air storage vessel at a pressure of 4,000 psi. It was built in five sections, flange bolted together and is supported on a fixed support at one end and on intermediate rollers at approximately 125 foot centers.

Expansion joints of the type shown in Fig. 11 are used on several installations in South America. The expansion joints have been designed to provide for pulling up an entire penstock section for a distance of approximately 12" by means of long pull-up bolts to facilitate assembly in rough terrain with steep gradients. The long pull-up bolts are substituted for the gland bolts and

3. "Design of Large Pipelines," by Herman Schorer, Transactions American Society of Civil Engineers, 1931.
4. "Welded Steel Penstocks," design and construction by P. J. Bier, Engineering Monographs No. 3, United States Department of the Interior Bureau of Reclamation, July 1949.

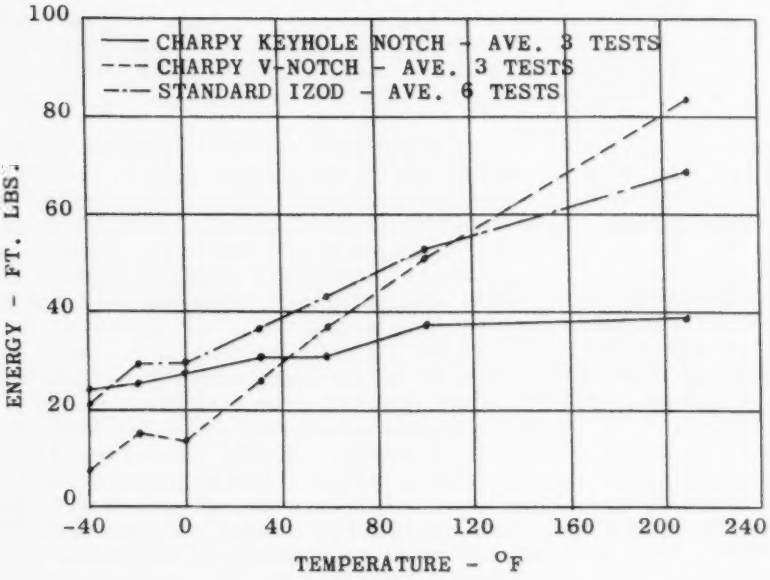


FIG. 9 IMPACT TESTS - TEMPERATURE TRANSITION CURVES OF CARBON STEEL FORGED FLANGE MATERIAL OF ASTM-A105 GRADE II MODIFIED SPECIFICATION.

LAGES "C" PENSTOCK FOR RIO DE JANEIRO T. L. & P. CO. NOTE EXPANSION JOINT AT RIGHT IMMEDIATELY BELOW CONCRETE ANCHOR BLOCK AND LUBRICATED SADDLE SUPPORTS.

FIG. 10

inserted through the flange at the up-stream end of the sleeve. The housing, sleeve and gland are machined after stress relieving to close tolerances with maximum clearances of 1/16" at the gland joint.

The standard length of multi-layer pipe sections as they come from the wrapping machine is 20 feet. The diameter of multi-layer pipe is limited only by shipping limitations and manufacturing equipment. Multi-layer penstock sections having an inside diameter of 35" or less may be made in 40' lengths using electric resistance flash-welded pipe for the inner shell. The plate material used for the inner shell and layers is in accordance with the modified ASTM-A212 specification given above.

Although multi-layer pipe is primarily intended to be used in an all welded penstock, there may arise the need for mechanical couplings. The ends of multi-layer pipe can be prepared for use of couplings as shown in Fig. 12. An extra layer of mild steel 10" wide is wrapped around the end of the pipe which is then welded shut at the end between the inner shell and over-wrap in order to make it water tight. The over-wrap, when machined to the required diameter, will insure accurate fit with the coupling. The seal weld is checked at the surface with a dye penetrant to insure that it is bottle tight.

The graph in Fig. 13 has been prepared to show the designer at a glance the useful range of multi-layer construction as it may apply to his specific needs. The curves are drawn on the basis of a hoop tension of 22,500 psi and for diameters from 2 to 12 feet which, however, are not necessarily the limiting sizes for multi-layer construction.

Fabrication

The inner shell plate is accurately layed out to specified dimensions after which the two joining edges are machine scarfed to required shape and size. Then the plate is rolled to the required diameter and the longitudinal seam is welded using one or a combination of the following methods:

1. Metallic arc fusion process with covered electrodes.
2. Submerged arc fusion process with bare or coated wire.
3. Electric resistance welding process.

The inner shell of 33" inside diameter and larger is first metallic arc welded at the outside. The seam is then submerged arc welded at the inside. Then the weld seam is chipped and ground flush at the outside and the shell is re-rolled in order to obtain a true diameter. Next, the seam is radiographed for its entire length in accordance with Section VIII of the ASME Code for Unfired Pressure Vessels. Finally, round-out rings are inserted tightly into each end and the shell is placed into the wrapping machine for application of the outer layers. In order to insure that the longitudinal edges of the layers will lay tight on the inner shell, both edges are crimped in a die for a distance of approximately 12" to the true outside radius of the inner shell. Hereafter the layer is rolled to the required radius and clamped onto the inner shell. The layer sections are tightened around the circumference by means of flat woven cables operated by hydraulic pressure and at the same time they are tack welded to the inner shell. On completion of this operation, the longitudinal seams are manually welded and the surplus weld metal is ground flush to insure metal to metal contact with the subsequent layers. Each successive layer is fabricated and wrapped in the same manner as the first layer. All the weld seams of the succeeding layer plates are staggered so that no seam

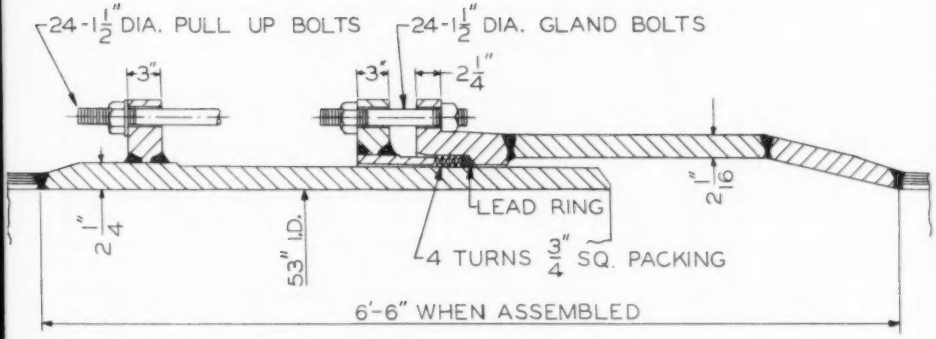


FIG. 11 TYPICAL EXPANSION JOINT FOR PENSTOCK UNDER 2350 FOOT STATIC HEAD. TEST PRESSURE 1750 PSI.

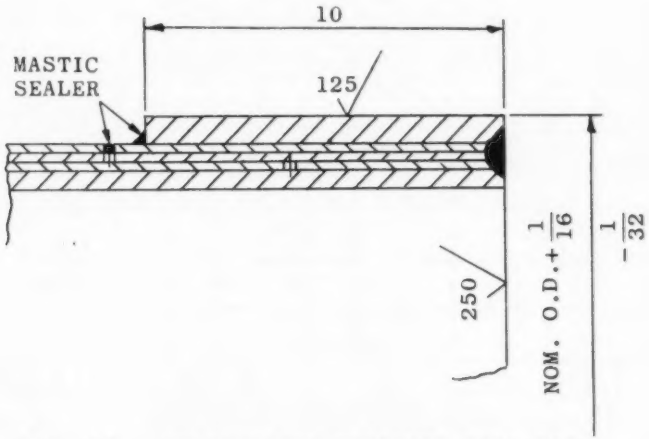


FIG. 12 MULTI-LAYER PENSTOCK PIPE FOR USE WITH MECHANICAL COUPLING

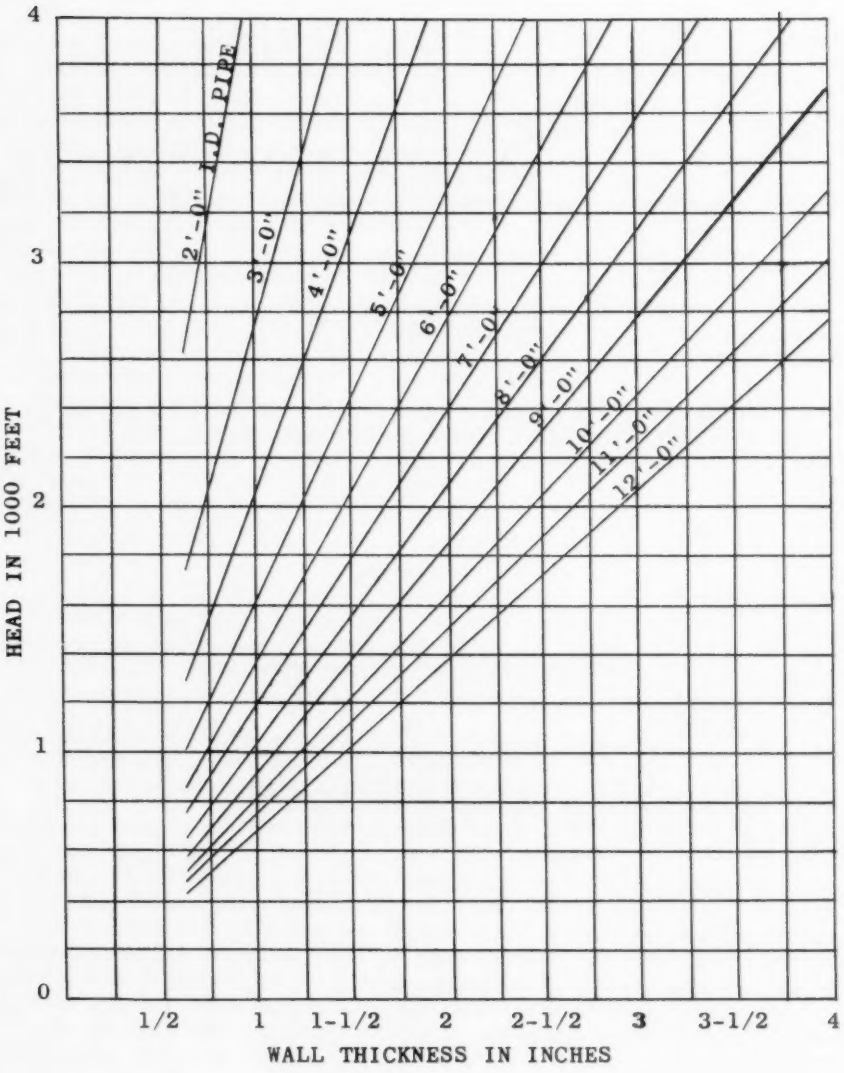


FIG. 13 USEFUL RANGE OF MULTI-LAYER CONSTRUCTION AT HOOP STRESS OF 22,500 PSI

is super-imposed on the seam of the previous layer as shown in Fig. 1.

Before wrapping, two 1/4" diameter vent holes are drilled 6" from each end in each layer plate that is to be welded for its entire thickness at the girth seams. Layers that are not welded at the girth seams do not need the vent holes because the ends are open to the atmosphere. At a fully welded girth joint such as shown in Fig. 8(b) and (d), the holes are pre-drilled through the first layer only. After the wrapping is completed, a 1/4" vent hole is drilled from the outside into the first layer which completes the path to the outside.

When two or more pipe sections are completed as described above, they are assembled to form a pipe of specified length which requires the welding of one or more circumferential joints. After scarfing, the pipe sections are brought into correct alignment and tack welded together. Then the assembly is placed on mechanical rollers and the inner shell girth seams are welded from the outside. After the outside welding of the inner shell is completed, the inside seam is back gouged and welded. The girth welds of the inner shell are then x-rayed to ASME Code standards. The outside layer welds are then completed. The depth of the weld, as previously explained, extends to the termination of the layer exceeding 60% of the total wall thickness. (See Fig. 8.)

The pipe is now subjected to the hydrostatic test. It is placed in a testing fixture designed especially for this purpose. Pressure is applied to produce a hoop stress equal to 80% of the minimum specified yield strength of the plate steel used and held for 15 minutes. If the test is satisfactory, the pipe is cleaned, painted and prepared for shipping in accordance with prevailing specification. For 6" from each end, the pipe remains unpainted because of subsequent field welding. Bends and other irregular pipes not suitable for testing in the fixture are tested by closing the ends with welded-on test heads.

In order to prevent rain water from entering the vent holes and between layers that are not welded at the girth joints, an especially prepared mastic compound is applied which has excellent adhesive qualities which it will maintain under severe weather conditions. This mastic compound has an asphalt base and is available in two grades for summer or winter application and is applied with a caulking gun.

The fabrication of solid wall portions of a penstock is in accordance with the requirements of Section VIII of the ASME Code for Unfired Pressure Vessels. It is not intended to describe this phase of the subject since it has been dealt with in numerous publications. The fabrication of multi-layer construction is in accordance with the applicable rules of the ASME Code but the product is not stamped with the ASME Code symbol because the Code does not cover this proprietary article in its rules.

Installation

The method of installing multi-layer pipe in the field is not much different and may be even simpler than that of single wall pipe because it has greater flexibility in handling fit-up problems. No preheat is necessary for welding the girth joints under normal conditions when the ambient temperature is above 50° F. Below 50° F localized preheat of 125° to 150° F is recommended. The usual method is to start erection from the bottom of each anchor and work in the up-stream direction. The shipment of pipe sections must be planned to suit the requirements in the field.

The first step is to establish vertical and horizontal center lines on the pipes which is followed by assembling and tack welding spacer shims along

one lip edge of the pipe being joined as described in the assembling and tack welding procedure in Fig. 14. A root gap of 5/32" is considered normal. Root gaps below 3/32" should be restored to the normal width by the use of an air hammer and chisel before welding the first layer. This is to assure complete penetration through the lips. If the root gap exceeds 1/4", a 3/16" thick x 1" wide chill strip of hot rolled mild steel is tack welded to the inside of the pipe. One bead is welded into the groove when the root gap is less than 9/16" and two weld beads are used when the root gap exceeds 9/16" as shown in Fig 15 (b) and (c) respectively.

The following sequence of welding between the tack welds is recommended. Referring to Figure 14 (b), the welding starts between station 3 and 9, followed by 4 to 7. If two welders are employed on one seam, both of these welds can be made simultaneously. The next step is to weld from station 8 to 3 and 10 to 4, then 5 to 1 and 6 to 2, then 11 to 1 and 12 to 2, 5 to 9 and 6 to 10, and finally 7 to 11 and 12 to 8.

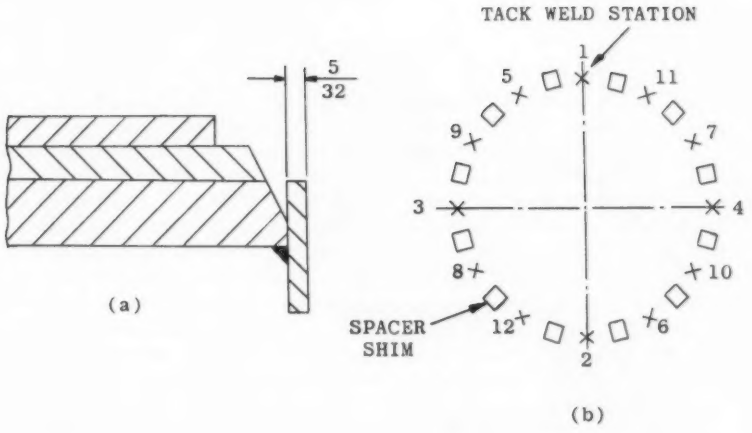
If, for any reason, it is preferred to start welding at the bottom half first, the same diametrical alternating principle should be followed as stated above. No welds must be made over the areas of spacer shims without first removing the shims. All vertical welds are to be welded from the bottom up. It is important that the first layer of the weld is not advanced more than 7" to 12" ahead of the second layer as the welding progresses. The remaining layers should be welded in such a manner so that the underlying layers are not more than 7" to 12" in advance of the succeeding layer at any time. A profile of a circumferential seam with two layers of weld is shown in Fig. 15 (a).

The lip at the inside must be removed and chipping, grinding, arc cutting or flame gouging may be used for preparation of the grooves for back welding. The groove must be carefully inspected for complete removal of any defects, lip edges and of all slag when arc cutting or flame gouging is employed. It is recommended that the welding groove be examined by magnetic particle inspection. The weld metal should be built up as a reinforcement at both sides of the pipe to 1/16" at the crown of the reinforcement. When the pipe has cooled to normal temperature, the mastic sealer previously described is applied to the joint to cover the open ends of the outer layers that are not welded.

The same procedure of assembly and welding applies to field joints where a single wall pipe is welded to a multi-layer pipe. The only difference is that the weld extends to the full thickness and is blend welded into the single wall pipe as shown in Fig. 6 (d). The electrodes recommended for field welding inside and outside are A. O. Smith Corporation SW-15 (E-6013), 5/32" diameter for the first pass and 3/16" diameter for all succeeding passes. Welding operators are to be qualified to meet all requirements of Section IX of the ASME Code.

The first multi-layer penstock was built for the Rio de Janeiro Tramway Light & Power Company in Brazil in 1945. It has three diameters, 76-3/4", 69-1/4", and 64-1/4". The maximum static head is 1,112 ft. The turbine is rated at 53,400 H.P. at 1,050 ft. net effective head.

The Serra penstocks No. 3, 4 and 6 for the Sao Paulo Light & Power Company in Brazil have been in operation since 1946, 1947 and 1949 respectively. The diameters are 61-5/8", 58-3/8", and 53", and two branch lines of 36" diameter. Each impulse turbine to which each penstock is connected is rated at 91,800 H.P. under 2,200 ft. net effective head. The maximum static head is 2,370 ft.



1. The 0 and 180° center lines are first established on pipes before assembling one pipe to the other.

2. Assemble and tack weld 5/32" x 1" x 1" spacer shims along one lip edge of pipe being joined, as shown in (a). Location of spacer shims is shown in (b).

3. Align pipes and tack weld at top and bottom center line at station 1 and 2 as shown in (b).

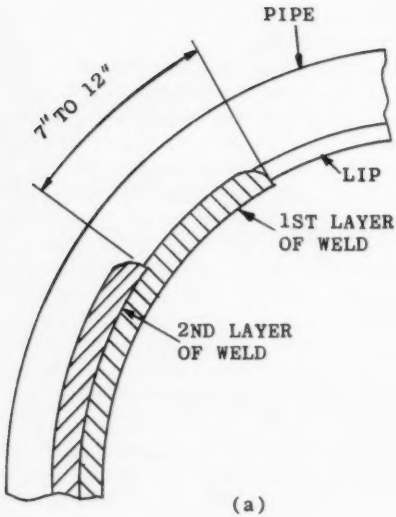
4. Complete the remaining alignment of pipe and tack welding, following the numerical sequences as indicated in (b).

5. Tack welds should be 4" or longer, two or more layers of weld. The welding conditions should conform to the cross section of the weld as shown in Fig. 15(d).

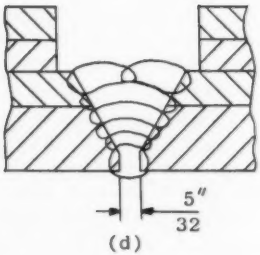
6. The scarfed edges of the pipes should be carefully examined for damaged lip edges, indentations and rust which may have occurred during shipment and be restored to their proper shapes and conditions before final assembly.

7. Taper both ends of all tack welds with an air hammer and chisel to assure a good weld junction.

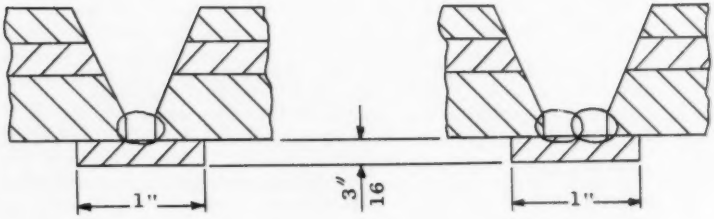
FIG. 14 ASSEMBLING & TACK WELDING PROCEDURE



(a)



(d)



(b) (c)

FIG. 15 ROOT GAP & WELDING DETAIL

A small diameter penstock for a maximum static head of 1,980 ft. was built from 26" O.D. 40 ft. long pipe for the Cementos El Cairo S.A. in Colombia. The lower half of the penstock is single-layer construction with one 3/16" and one 1/4" thick layer wrapped around a 3/8" thick pipe. The layers are not welded at the girth seams.

Wye Construction

For the lower end of the 53" diameter portion of the Serra No. 4, 5 and 6 penstocks, wyes of approximately 66 degree angle with 36" diameter branch lines were furnished. They operate under a static head of 2,370 ft. The wall thickness is 2-1/8" and the plate material is to ASTM-A212 Grade B fire box quality specification. A 3-1/2" thick splitter plate is welded through the middle of the crotch. Forged steel flanges with tapered hubs were welded to each opening. The mating pipes for the lower branches were provided with Van Stone type loose flanges to facilitate field erection. A pair of tapered dutchmen were used at each flange connection to take care of any possible discrepancy in distance between and misalignment of the flange faces. Each joining face was provided with a double set of 1/2" diameter "O" ring rubber gaskets in recessed gasket grooves to make sure that the joints will remain bottle tight. There are thirty-two 3" diameter bolts at the up-stream flange and 2-1/4" diameter bolts at each of the branch connections. The bolting was furnished to ASTM-A193 B7 specification and with ball and socket washers. All weld seams of the wye were x-rayed or magnafluxed and the assembly was stress relieved at 1150° F for 3-1/2 hours. It was then given a hydrostatic test of 1,860 psi after a preliminary hammer test while under a pressure of 1,500 psi. The entire wye was embedded in concrete to become part of the last anchor block above the power house.

A problem presented itself in the design of the branches for the lower end of the Kemano penstocks for the Aluminum Company of Canada. There are two pressure shafts of 11'-0" inside diameter at the down-stream end. Each terminates in a 90 degree wye having two branch lines of 7'-9" inside diameter. Each of the branch lines in turn terminates in a 45 degree wye having branch lines of 5'-6" inside diameter.

For the design pressure of 1,235 psi, an allowable stress of 23,100 psi was specified for plate material to ASTM-A212 specification. The first design for the large wye was planned using a three-plate reinforcement of I-beam cross section. However, in order for the reinforcing plates to be within reasonable thickness, they became of such size as to exceed the shipping clearance for a completely shop fabricated unit.

A spherical wye of 12'-3" inside diameter was proposed with the result that the maximum shipping width was only 12'-10 3/8". The spherical portion of the wye was made from 3-11/16" thick plate formed in twelve gore sectors and one cap to a 73-1/2" spherical radius. Only enough stock was provided at the up-stream end for scarfing the opening for a butt joint to the 11 ft. inlet pipe also of 3-11/16" wall thickness. The two openings for the 7'-9" branch outlets of 3-7/16" wall thickness were accurately layed out and flame cut. The pipes were inserted and welded from the inside against a chill ring, which was later removed and back welded. All seams were x-rayed or gamma-rayed. Although the spherical portion of the wye was thicker than needed to carry the pressure load within the allowable design stress, it had to sustain the additional loading of the discontinuity at the openings. Additional collars

equal to a cross sectional area of approximately 40 sq. in. were welded to the sphere and around each pipe. These welds were all magnafluxed. At the inside of the sphere, in the crotch between the two branches, a contoured plate was welded to provide more reinforcement.

After completion, the wye was stress relieved at 1150 °F for six hours followed by the hydrostatic test in which strain gage readings were taken at various points at 100 psi gage pressure intervals up to the maximum test pressure of 1,435 psi. The strain gage test indicated a uniform stress distribution and the measured stresses very nearly approached those calculated.

Fig. 16 shows a view of the wye under test. The white bands indicate white wash applied to critical areas. No local yielding could be detected at the white washed areas. The shipping weight of the wye was 45 tons.

The 7'-9" diameter wyes for the lower branches were reinforced with three horse shoe type integral harnesses. The high pressure of 1,235 psi in conjunction with the relatively large diameters resulted in thick reinforcing members. The plate material was to ASTM-A212 Grade B specification. The "harness bow" for the acute crotch of the wye is a forging to ASTM-A105 Class 2 specification with modifications given above. The construction detail is shown in Fig. 17 and Fig. 18 shows the completed wye under hydrostatic test. The test pressure was 1,435 psi. Strain gage readings taken during the test showed that the stresses were well below the maximum allowable stress. The shipping weight of the wye was 37 tons. All wyes were provided at each end with short stubs made from steel to ASTM-A201 Grade A specification to provide lower carbon materials in these heavy sections for facilitating field welding.

In 1953 work began on the distribution manifold No. 1 for the Cubatao underground plant for the Sao Paulo Light & Power Company in Brazil. The penstock is a steel lining imbedded in concrete surrounded by rock. The lower portion has an inside diameter of 9'-8 1/8" and is placed in a horizontal tunnel 230 ft. long. Although it is surrounded by concrete, it has a transition in wall thickness from 1" to 1-15/16" in 1/16" increments for its length. Because of dubious strength of the rock, the manifold has been designed so that the steel will sustain the full hydrostatic pressure although it too is imbedded in concrete. A plan view of the transition and the manifold, together with a table of thicknesses and weights of the various pipe sections, is shown in Fig. 21. There are six branches of 63" inside diameter leading from five 60 degree wyes. Fig. 19 shows a view of the wyes after their completion.

The single wall plate material for the transition section, wyes and branches is to a modified ASTM-A225 specification in order to permit high working stresses with minimum wall thickness. The diameters of the five wyes range from 114-3/16" to 76-3/4". The plate material for the reinforcing harnesses for the wyes was to ASTM-A212 Grade B specification, while the harness bows were forgings to a modified ASTM-A105 Class 2 specification. The largest wye was subjected to a hydrostatic shop test of 1,275 psi with a corresponding hoop stress of 38,500 psi at the largest diameter of the wye. This stress is equal to 80% of the specified minimum yield strength of the material for the shells.

Fig. 20 shows a view of one of the six flanged pipes that connect to the valves in the power cavern. The pipe has a reducer from 63" to 47-1/2" I.D. ending in a flange 76-1/2" outside diameter and 9-1/16" thick.

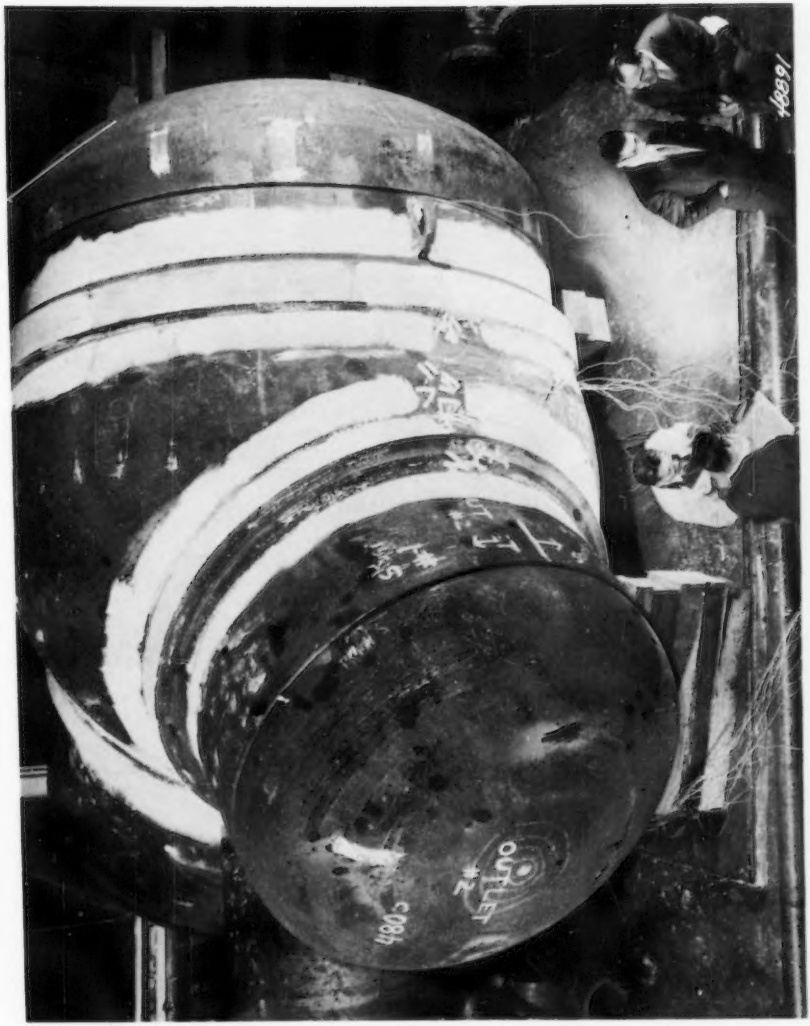
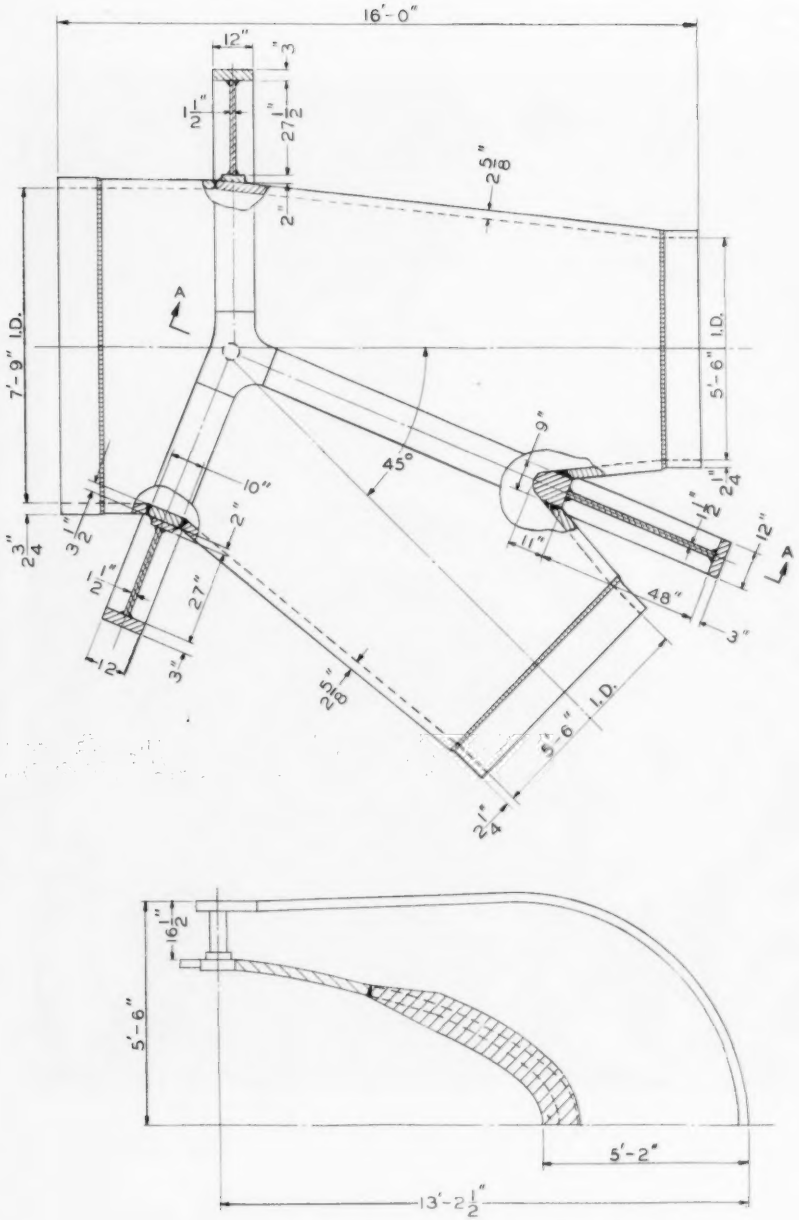


FIG. 16 HYDROSTATIC TEST OF SPHERICAL WYE



SECTION A-A

FIG. 17 WYE DESIGNED FOR 1235 PSI PRESS.
TESTED AT 1435 PSI PRESS.

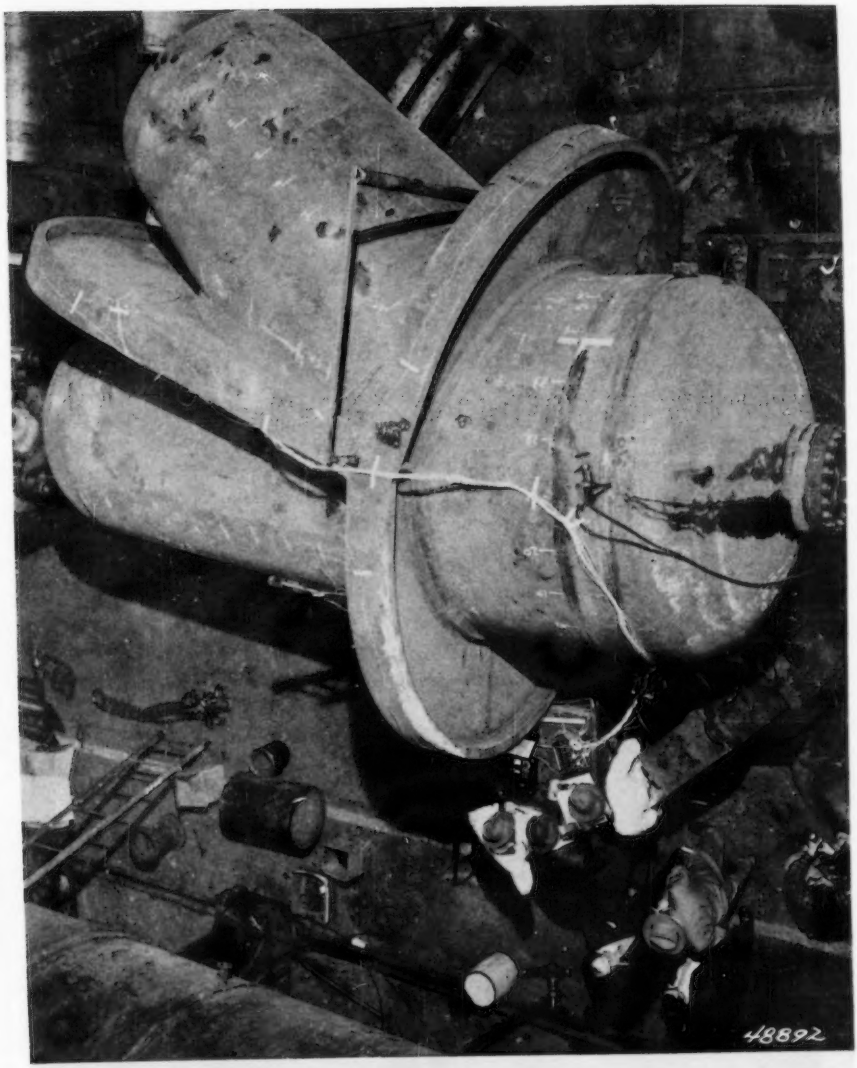


FIG. 18 45° WYE UNDER HYDROSTATIC TEST

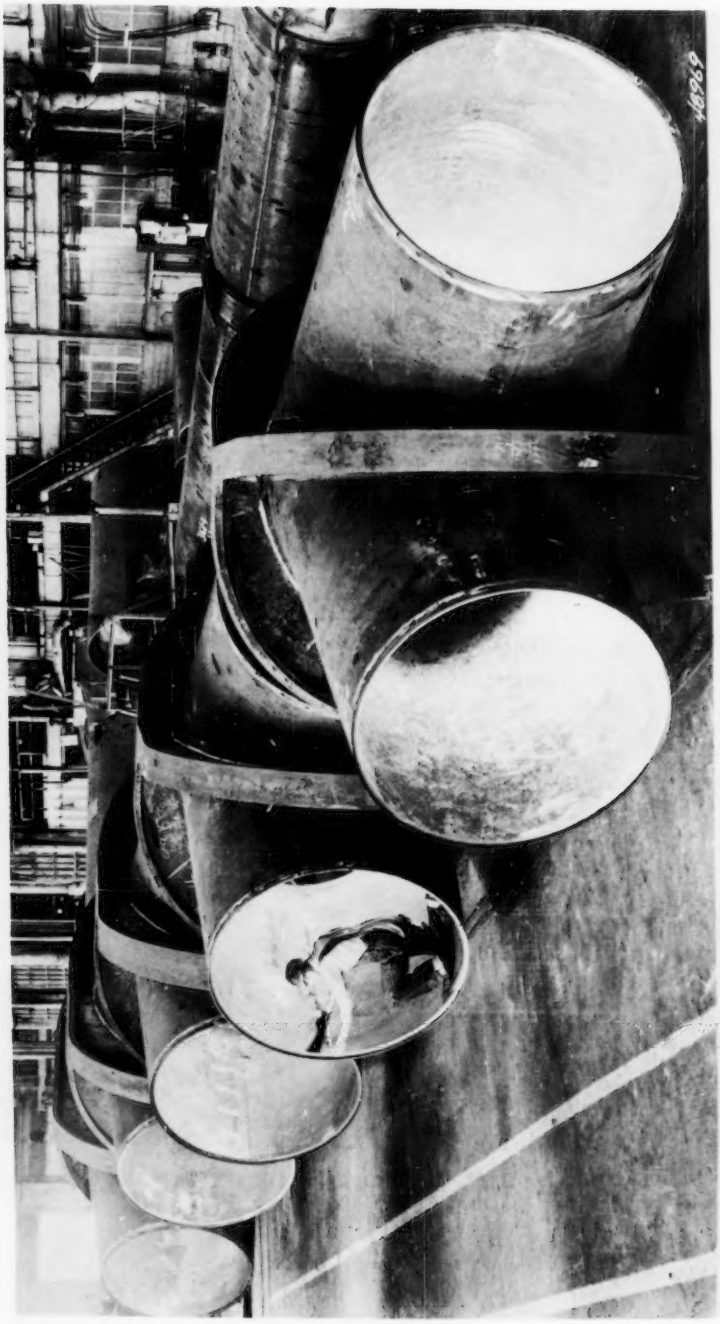


FIG. 19 FIVE WYES FOR CUBATAO DISTRIBUTION MANIFOLD

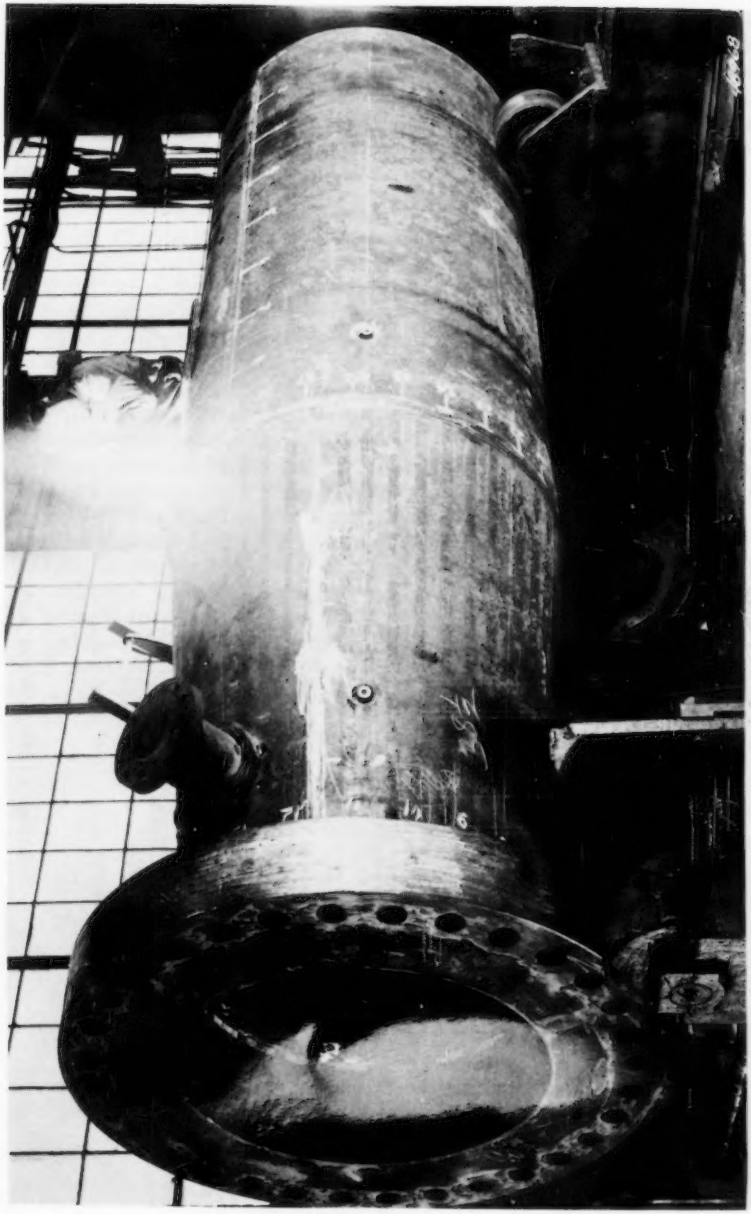
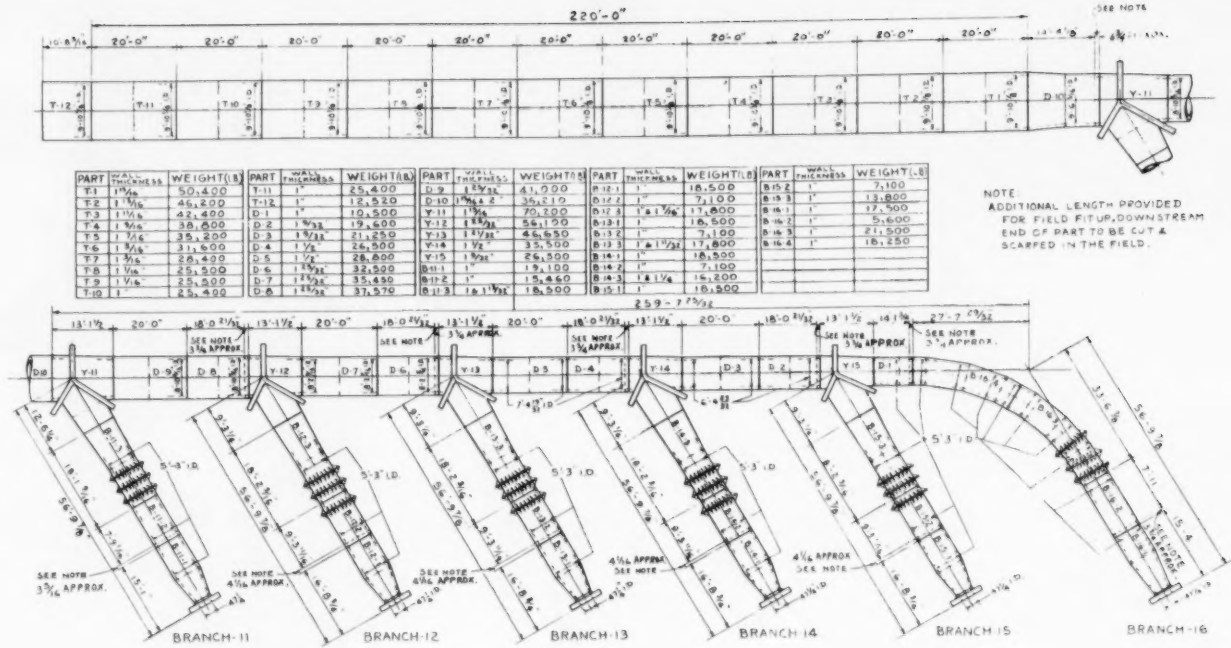


FIG. 20 ONE OF SIX FLANGED ENDS FOR CUBATAO MANIFOLD



PART	WALL THICKNESS	WEIGHT(LB)
T-1	1 15/16	50,400
T-2	1 15/16	46,200
T-3	1 15/16	42,400
T-4	1 9/16	38,600
T-5	1 7/16	35,200
T-6	1 5/16	31,500
T-7	1 3/16	28,400
T-8	1 1/16	25,500
T-9	1 1/16	25,500
T-10	1	25,400

PART	WALL THICKNESS	WEIGHT(LB)
T-11	1"	25,400
T-12	1"	12,520
D-1	1"	10,500
D-2	1 9/32	19,600
D-3	1 5/32	21,250
D-4	1 1/32	26,500
D-5	1 1/32	28,800
D-6	1 3/32	32,500
D-7	1 5/32	35,490
D-8	1 5/32	37,570

PART	WALL THICKNESS	WEIGHT(LB)
D-9	1 23/32	41,000
D-10	1 11/16 & 2	36,210
Y-11	1 15/16	70,200
Y-12	1 29/32	56,150
Y-13	1 17/32	46,650
Y-14	1 1/2	33,500
Y-15	1 9/32	26,500
B-11-1	1"	19,100
B-11-2	1"	15,460
B-11-3	1 & 1 11/32	18,900

PART	WALL THICKNESS	WEIGHT(LB)
B-12-1	1"	18,500
B-12-2	1"	7,100
B-12-3	1 & 1 7/16	17,800
B-13-1	1"	18,500
B-13-2	1"	7,100
B-13-3	1 & 1 11/32	17,800
B-14-1	1"	18,500
B-14-2	1"	7,100
B-14-3	1 & 1 1/4	15,200
B-15-1	1"	18,500

PART	WALL THICKNESS	WEIGHT(LB)
B-15-2	1"	7,100
B-15-3	1"	13,800
B-16-1	1"	17,500
B-16-2	1"	5,600
B-16-3	1"	21,500
B-16-4	1"	18,250

CUBATAO DISTRIBUTION MANIFOLD NO. 1

FIG. 21

CONCLUSIONS

Up to the present time, multi-layer penstocks have been built for above ground installation. However, this construction could be applied also for below ground installation in an open tunnel where the pipe is not imbedded in concrete. The necessity and desirability of venting the inner shell to the atmosphere would complicate the installation if the pipe were to be grouted in concrete. From the foregoing treatise, the following conclusions may be reached:

1. An all welded penstock is desirable for the following reasons:
   a) Its smooth interior will appreciably reduce the head loss.
   b) It will remain bottle tight and reduce the cost of maintenance.
2. Multi-layer construction, which is only limited by transportation and handling facilities, offers a safe method for an all welded penstock 5/8" thick and over. Transition temperatures are inherently lower in thin wall material from which multi-layer pipes are fabricated and provide less susceptibility to brittle fracture.
3. With no upward restriction in wall thickness and with higher allowable design stresses, it is possible to consider one large diameter multi-layer penstock with manifolding at the power house in place of two or more parallel lines of smaller diameter of single wall construction.
4. Multi-layer pipes, being lighter than conventional single wall units, reduce transportation and handling charges.
5. Lesser amount of field welding at circumferential joints with total elimination of stress relieving, irrespective of wall thickness and without endangering the safety of the installation, results in additional substantial savings in erection time and cost.
6. If the thickness required for a single wall penstock exceeds 3/4", multi-layer construction may offer advantages in the over-all economy of the installation.

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by Bert J. Blewitt 1346-8
by C. W. Bary and W. T. Brown 1346-10

Water-Hammer Design Criteria, by John Parmakian. (Proc. Paper 1216. Prior discussion: none. Discussion open till September 1, 1957.)
by Carroll E. Withers 1346-13

Note: Paper 1346 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, PO 4, August, 1957.

Discussion of
"ARCH DAMS: DESIGN AND OBSERVATION
OF ARCH DAMS IN PORTUGAL"

by M. Rocha, J. Laginha Serafim, and A. F. da Silveira
(Proc. Paper 997)

M. ROCHA,* J. LAGINHA SERAFIM** M. ASCE, and A. F. DA SILVEIRA.***—The writers thank Robert E. Glover and Fred A. Houck for the attention and appreciation shown their paper.

In the discussion of Robert E. Glover and M. D. Copen's paper "Trial Load Studies for Hungry Horse Dam" (Proc. Paper 960) and in the closing discussion of the paper "Model Tests, Analytical Computation and Observation of an Arch Dam" (Proc. Paper 696) presented by the writers, an opportunity was presented for giving their opinion on the advantages and setbacks of the trial-load method. It is not intended that we should repeat the reasons already given; the trial-load method can supply the same results as a model test, when the dam is not complex, that is, when both shape irregularities and heterogeneity in the properties of the materials are absent. However, as the designer has to take into account in the majority of cases, irregularities and heterogeneities chiefly in the foundation rock, he has no tool to preview the behaviour of the structure unless he follows Mr. Glover's suggestion at the end of his discussion, obtaining an overdesigned structure. The advantage of experimental methods is that they can be applied both to simple and complex cases, always with the same accuracy and at a cost lower than the completetrial-load calculation, provided that adequate techniques are used. At present, with the modern high precision testing techniques using small models, a model test costs less than a calculation by the complete trial-load method, as the writers could verify based on the numbers given by Mr. Houck. Note, for instance, that a complete adjustment supplies information on one reservoir level only, whilst informations on the different reservoir levels—what sometimes is of very considerable interest—can be obtained without any change whatsoever in the arrangement of the measuring apparatus. Additionally, it is possible to study various solutions on the same model, if care is taken to begin by the thickest shape. The writers believe that not only the model tests carried out at the Bureau of Reclamation were the first of their kind to be made but also that the measuring techniques were then insufficiently developed, from which resulted a great waste of time and money and a lack of reliance in the results. At present, in Portugal, the preliminary design justified by analytical calculations supplies the first dam shapes to be tested

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on model, whilst the final shapes, that is the design proper, are based, exclusively on the model tests.

The writers are glad to congratulate Mr. Glover on the interesting synthesis presented with regard to the need to make all the adjustments in the calculations by the trial-load method and the influence of the tangential and twist adjustments on the stress distribution.

Discussion of
"A SUPPLEMENTAL NOTE ON VALUATION AND DEPRECIATION"

by Maurice R. Scharff
(Proc. Paper 1184)

PAUL H. JEYNES.[1]—In this era of high income-tax rates, when income taxes for a company having no bonded debt may be expected to exceed the earnings available for dividends, it is surprising that the literature on valuation and depreciation contains so little on this subject beyond Mr. Scharff's contributions.

The following remarks deal with the problem of anticipating future changes in annual expenses, and the effect of adopting different methods of depreciation for book purposes and for tax purposes, respectively, as permitted by the present law.

Rate of return on capital investment is a "just-as-soon" concept. If the investor finds i% return acceptable, he would just as soon have (1) one dollar in hand, or (2) i cents annually, forever, where i is the acceptable rate of return in percent. This definition permits making calculations of present value in terms of annual payments over any stipulated period. It also permits expressing non-uniform annual quantities in terms of their levelized annual equivalents, as Mr. Scharff has done.

So far as annual capital-recovery costs are concerned (i.e., return on investment plus depreciation), it is most convenient to express them as an equivalent levelized percentage, applicable to net investment each year throughout the lifetime of that investment.(1) This percentage does not change when equipment becomes relatively obsolete, or is demoted from base-load to standby service. It does not cease until the equipment is retired and finally withdrawn from the capital accounts.

Related income taxes then remain a constant percentage of that same investment throughout that same lifetime, provided it can be anticipated that:

1. The same depreciation methods will continue to be used throughout for books and taxes, respectively,
2. The same statutory tax rate will remain in force, and
3. The same percentage of annual return will be represented by deductible interest on debt.

If there is any reason to expect changes in any or all of these factors with passage of time, there is no alternative but to recognize the effect on annual revenue requirements. This same observation applies to changes in annual expense such as operation, maintenance, and other taxes. There is no magic in the straight-line assumption as to future rate of change in such costs. It can be defended only for want of anything better.

Having made whatever assumptions appear best, there is no question as to the calculation of present value. The present worth of the lifetime revenues

1. Eng. Economist, Pub. Service Electric & Gas Co., Newark, N. J.

required to cover all annual costs, including the acceptable return on investment, is:

$$C + \frac{Ce}{i + \beta_i d}$$

where C = initial investment
i = acceptable rate of return, decimally
$\beta_i d$ = the depreciation annuity, decimally
e = annual expenses (operation, maintenance, taxes) in percent of C.

The general expression for annual income taxes in percent of C (i.e., the levelized lifetime figure), per the present law, and in terms of the depreciation methods used for book purposes and tax purposes respectively, is readily developed from the statutory definition of taxable income. It may be described in four steps, with a fifth step necessary if the company "normalizes" income after having adopted the liberalized treatment of depreciation now permitted:

1. Levelized annual revenue requirements, in percent of C, are estimated as $i + \beta_i d + T$ (= return plus depreciation annuity plus income tax).

2. Levelized "depreciation for tax purposes" (= d') is calculated and deducted.

3. Levelized "deductible interest on debt" is calculated and deducted.

4. The remainder of revenue, representing taxable income, including income tax is multiplied by $\frac{t}{1 - t}$, where t = the statutory tax rate, decimally.

That is:

T = t times "revenue less deductibles," by law

$= t (i + \beta_i d + T - d' - \text{deductible interest})$

$= t (i + \beta_i d - d' - \text{interest}) + tT.$

Thus:

$T (1 - t) = t (i + \beta_i d - d' - \text{interest}).$

and

$$T = \frac{t}{1 - t} (i + \beta_i d - d' - \text{interest}).$$

The levelized value of d' may be calculated for any stipulated method of depreciation and any given life, type of retirement dispersion, and net salvage value. Values of $\beta_i d$ appear in Reference (1).

When income is "normalized," taxable income expressed in the manner of item 4 must be reduced by the amount appearing as "provision for deferred federal taxes on income." For example, assuming that "straight-line" depreciation is used for book purposes (where $_o d = \frac{1}{\text{life}}$), and d' is the

depreciation for tax purposes, this credit becomes t (d' - $_{o}$d).

Two brief illustrative examples follow, describing the calculations for:

A. "Straight-line" depreciation for books and taxes

B. "Straight-line" for books; sum-of-years'-digits for taxes; income normalized.

Mr. Scharff has called attention to a principle that is completely sound. Anticipated income taxes do indeed affect present value in a manner that can be calculated if all of the variables involved can be estimated. The calculations, although rather lengthy, are quite definitive. Further tax rates, future percentage deductible interest, probable lives, and such variables remain estimates based on judgment, and there is no substitute for that judgment.

For brevity, all of the accompanying calculations have been made in terms of total lifetime of an investment. It is perhaps unnecessary to add that the value at any later date is the present worth at that time in terms of the future costs over the remaining life expectancy.

REFERENCE

1. "The Depreciation Annuity," by Paul H. Jeynes. AIEE Transactions, Power Apparatus and Systems, February 1957. Number 28, page 1398.

Table I

Levelized Lifetime Annual Income Taxes

For plant having 10-year life, no retirement dispersion, zero net salvage. 6% return. 50% of capitalization in bonds at 3% interest. 52% statutory tax rate.

A. "Straight-line" Depreciation for Books and Taxes

Capital-recovery factor = i + $_{i}$d 13.5868%
Less book depreciation = $_{o}$d 10.0000
Taxable income plus interest less tax = 3.5868% = return on gross plant.

Deductible interest = $\frac{Bb}{i} = \frac{0.50 \times 3.0}{6.0}$ = 25% of return on gross plant.

In percent of investment = 0.25 x 3.5868 = 0.8967%
Taxable income after tax = (i + $_{i}$d - $_{o}$d) less 0.8967
= 3.5868 - 0.8967 = 2.6901%

Tax = $(\frac{0.52}{1.00-0.52}) \times 2.6901 = 2.9143\%$

Expressed as a formula:

$$T = (\frac{t}{1-t})(i + {}_{i}d - {}_{o}d)(1 - \frac{Bb}{i})$$

B. "Straight-line" Depreciation for Books; SYD for Taxes; Income Normalized

It is necessary first to calculate the level annuity equivalent to the non-uniform SYD charges, as follows:

Year	Book Charge		Present Worth Factors	Present Worth
1	$\frac{10}{55}$ x 1,000 =	181.818	0.9433 962	171.5264
2	$\frac{9}{55}$ x 1,000 =	163.636	.8899 964	145.6355
3	etc.	145.455	.8396 193	122.1268
4		127.273	.7920 937	100.8121
5		109.091	.7472 582	81.5191
6		90.909	.7049 605	64.0873
7		72.727	.6650 571	48.3676
8		54.545	.6274 124	34.2222
9		36.364	.5918 985	21.5238
10		18.182	.5583 948	10.1527
	Total	1 000.000		799.9735

Equivalent annuity = $(i + {}_id) \times \frac{799.9735}{1,000} = 10.8691\% = d'$

Capital-recovery factor = $i + {}_id$ = 13.5868%

Less book depreciation = ${}_od$ = 10.0000

Less provision for deferred taxes = $t(d' - {}_od)$

= 0.52(10.8691 = 10.0000) = 0.4519

Taxable income plus interest less tax = 3.1349%

Interest = $\frac{Bb}{i} \times 3.1349 = \frac{0.50 \times 3.0}{6.0} \times 3.1349 = 0.7837\%$

Taxable income after tax = $i + {}_id - d'$ less 0.7837%

= 13.5868 - 10.8691 - 0.7837 = 1.9340%

Tax = $(\frac{0.52}{1.00-0.52}) \times 1.9340 = 2.0952\%$

Expressed as a formula:

$$T = (\frac{t}{1-t}) \left[(i + {}_id - {}_od) - t (d' - {}_od) \right] (1 - \frac{Bb}{i}) - t (d' - {}_od)$$

BERT J. BLEWITT.[1]—Mr. Scharff is to be congratulated for another contribution to the field of engineering economy. In this paper he develops a methodology for determining the justifiable investment in an existing public utility facility. The principal feature of his approach involves the determination of and the equating of the annual costs of an existing facility with a new facility of the same capability. The author has demonstrated the effect on the justifiable investment of (1) the exclusion of various elements of total annual costs, and (2) the variation in the behavior of various elements of total annual costs.

This type of problem has special application in the day-to-day operations of a public utility. The determination of the proper purchase and selling price of facilities in place, and the evaluation of the relative economies between replacing and repairing are examples of problems of this general nature.

The author, presumably in the interests of simplicity, has not considered the effects of retirement dispersion in any of his 8 cases. One other awkward factor is encountered in all of the formulas in that A_c and A_e (the annuities for periods L_c and L_e, respectively) cover different periods. Freedom of choice as to the type of replacement is regained earlier under one alternative than under the other.

1. Eng., Public Service Electric and Gas Co., Newark, N. J.

There are several methods for dealing with the latter problem. One is to assume that the existing facility will be replaced after period L_c by one similar to the present economic alternative. This is implicit in Mr. Scharff's solution, which tacitly assumes zero differentials beyond period L_c.

However, if the present alternative is superior as the result of advances in the art, it is not unreasonable to assume that future advances in the art can be anticipated. Existing equipment, replaced after L_c years, will realize these future savings sooner than will be the case for the alternative. Mr. Scharff then underestimates the competitive value of V_c.

On the other hand, for equipment less subject to such effects (such as poles and wire), the problem may be resolved by assuming that future replacements will in each case be made "in kind," respectively. Present worths of annual revenue requirements may then be calculated to eternity, as illustrated below. Since present-worth factors to eternity are substantially the same as for 60 years, this calculation is a reasonable approximation of conditions over one or two life cycles for most types of utility plant.

Under the assumption of "replacement by like kind" and determining the level annual values of O_e, P_e and U_e in percent of first cost E and level annual values of O_c, p_c and u_c in percent of the justifiable investment V_c, the algebraic expression may be set up as follows:

$$V_c \left[(R + {}^{\alpha}_{i}d_c) + \frac{I}{1-I}\left(R + {}^{\alpha}_{i}d_c - \frac{1}{L_c}\right)\left(1 - \frac{Bi}{R}\right) + O_c + p_c + u_c\right]$$

$$= E\left[(R + {}^{\alpha}_{i}d_e) + \frac{I}{1-I}\left(R + {}^{\alpha}_{i}d_e - \frac{1}{L_e}\right)\left(1 - \frac{Bi}{R}\right) + O_e + p_e + u_e\right]$$

where ${}^{\beta}_{i}d_c$ and ${}^{\beta}_{i}d_e$ are the depreciation annuities in percent of V_c and E respectively based on average lives of 20 years for the existing facility and 35 years for the competitive facility both at 6% return and having GC type dispersion and 0% net salvage in all cases.

Admittedly this approach involves an estimate of O_c, p_c and u_c in percent of V_c (the unknown). However if absolute values of these figures are available the level annual equivalent at R% can be calculated. In this case O_c, p_c and u_c are taken outside of the brackets. The same treatment, of course, can be applied to O_e, p_e and u_e. This does not eliminate the desirability of expressing these terms as a percent of first cost based on experience and informed judgment.

The equation above, which is an expression for equal annual costs, can be rewritten:

$$V_c = \frac{E\left[(R + {}^{\alpha}_{i}d_e) + \frac{I}{1-I}\left(R + {}^{\alpha}_{i}d_e - \frac{1}{L_e}\right)\left(1 - \frac{Bi}{R}\right) + O_e + p_e + u_e\right]}{\left[(R + {}^{\beta}_{i}d_c) + \frac{I}{1-I}\left(R + {}^{\beta}_{i}d_c - \frac{1}{L_c}\right)\left(1 - \frac{Bi}{R}\right) + O_c + p_c + u_c\right]}$$

where $E = \$1{,}000{,}000$

$R = 6\%$

${}^{\beta}_{i}d_e = .0191$

${}^{\beta}_{i}d_c = .0379$

L_e = 35 yrs.
L_c = 20 yrs.
B = 50%
i = 3%
O_e = 10% (assumed)
O_c = 15% (assumed)
p_e = 3 1/2% = .035
p_c = 5 1/4% = .0525
u_e = 1/4% = .0025
u_c = 3/8% = .00375
I = 50%

$$V_c = \frac{\$1{,}000{,}000\ \left[(.0791) + (.0791 - .0286)(.75) + .10 + .035 + .0025\right]}{\left[(.0984) + (.0984 - .0500)(.75) + .15 + .0525 + .0038\right]}$$

$$= \$1{,}000{,}000\ \frac{(.0791 + .0384 + .1375)}{(.0984 + .0369 + .2063)}$$

$$= \$1{,}000{,}000\ \frac{(.2450)}{(.3416)} = \underline{\underline{\$717{,}213}}$$

C. W. BARY[1] and W. T. BROWN.[2]—The author is to be congratulated on a clear and concise presentation of his subject showing, step by step, the effects of the various factors that enter the evaluation of the justifiable investment in an existing facility of a utility. These same factors also enter, in a similar manner, in the development of proper fixed charge rates to be used in economic comparison of alternate plans to meet the need of a growing utility. The writers have investigated this subject quite extensively and presented the findings in a recent paper before the American Institute of Electrical Engineers on "Some New Mathematical Aspects of Fixed Charges."(1) As a result of this investigation, the following discussion is offered as a helpful supplement to the author's excellent paper.

On a utility system, whose invested capital funds are ordinarily composed of debt and equity money, the discount rate, as applied to the conventional levelizing funds conceptionally employed in the present worth calculations, should be the estimated net annual cost of such capital funds to the utility. In the past, many investigators have approximated this discount rate by arbitrarily assuming that it is equal to the estimated average return rate required by the invested capital funds. But, obviously, the actual net annual cost of such funds to the utility is considerably less than the average return rate, due to the inherent and appreciable income tax credit directly associated with the indebtedness portion of such capital funds. This inherent deductible feature in establishing a discount rate for present worth calculations, is not to be confused with its proper application in the determination of the income tax component of the fixed charge rate.

1. Philadelphia Electric Co., Philadelphia, Pa.
2. Philadelphia Electric Co., Philadelphia, Pa.

The true discount rate applicable to all present worth computations is, therefore, equal to (R-IiB), using the author's symbols, as clearly demonstrated in Tables VIII and IX in the Closure of the recent paper.[1]

If, for example, R = 6%, I = 52%, i = 40% and B = 0.5; then:

$$(R = IiB) = 6\% - 52\% \times 0.04 \times 0.05 = 6\% - 1.04\% = 4.96\% \text{ or say } 5\%.$$

The present worth of a series of constant annual charges for a period of 50 years at a discount rate of 5% is about 16% higher than at a discount rate of 6%. In an economic comparison of alternate plans, the annual charges are usually not constant, and hence the present worth differential due to the employment of a 5%, as compared to a 6%, discount rate could be considerably more or less than the 16% indicated in the constant annual charge example described hereinbefore. Hence, for practical purpose, the significance of employing either the computed 5%, or the arbitrarily selected 6% discount rate on the magnitude of the present worth differential will depend upon the plan under consideration.

This discussion is confined to the operation of the discount rate in the region of investment compound growth rates of utility plants equal to or below the computed discount rate. The writers' paper[1] also contains findings pertaining to compound growth rates beyond that region.

REFERENCE

1. AIEE TRANSACTIONS, Volume 76, Part III, June, 1957, Pages 230-248.

Discussion of
"WATER-HAMMER DESIGN CRITERIA"

by John Parmakian
(Proc. Paper 1216)

CARROLL E. WITHERS.[1]—This paper presents a very logical procedure for including the effects of the transient hydraulic conditions which occur with any change in the velocity of any fluid being transported in any pipeline. The criteria presented has been proven by many successful pipe installations, many field tests, and, believe it or not, by a failure or two.

In the very highly competitive economic system of today there are several materials which will serve equally well for transporting water. It is quite often desired to have the most economical installation possible—not an installation influenced by politics, salesmen's tall tales, advertising or the personal prejudice of any individual. In such a case the only way alternate materials can be competitive, is for the design criteria to be identical for all materials. What has happened in the past is that with some pipe materials other than steel plate design, the water-hammer effects have been lumped in with other "factors of ignorance" concerning the capabilities of the pipe, and as a result, the overall factor of safety is unknown.

It is quite evident from this paper that the design of penstocks and pump discharge lines must follow different criteria. This is important to remember as the design problems are not the same. The location of surge tanks, for instance, presents a different problem in each case.

One point that could be slightly confusing is the last two conditions of operation. The "emergency condition" is sufficiently clear. However, the "emergency condition of operation not considered as a basis for design" should be renamed, and if the stress due to these conditions are kept below the ultimate strength of the material, then it is considered as a basis of design. This point could stand a little more discussion from the author.

The portion of the design criteria for turbine penstocks has been in use for over 8 years and has proven satisfactory in all cases. The pump discharge line design criteria has also been proven in many successful designs. However, there has been some discussion about including power failure to the pump motors as a normal operation problem. Not too long ago the writer was working in a pumping plant during a very severe electrical storm. In about a 5 hour period there were 7 shutdowns due to power failure. This certainly substantiates power failure to the pump motor as a normal operating condition.

Under emergency conditions of pump operation is this statement: "If a check valve is used to shut off the return flow through the pump, the check valve closure of one unit will be assumed to be delayed, and to occur (close) at the time of maximum flow." About 2 years ago, the writer had a very unhappy experience with a malfunctioning check valve. The pumping plant

1. Design Engr., Penstock and Steel Pipe Section, Mech. Branch, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

involved had two pumps which were connected by a wye-branch to a single discharge line. There was a check valve next to the pump, and a gate valve ahead of the check valve. Each pump had a 250 gpm rating at 218 foot head. One of the check valves failed to close properly and a very large reverse flow was established back through the pump. After the reverse flow had occurred for some unknown length of time the check valve slammed shut. The force created by this closure was so great that it caused a complete failure in body of the gate valve thus flooding the pumping plant. Although this failure could not have been caused by water-hammer alone, it indicates what can happen due to malfunctioning of a check valve.

Mr. Parmakian has had published a book entitled "Water-Hammer Analysis" which should be available to every engineer that has to make water-hammer calculations.

Paper 1350 August, 1957 PO 4

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

BIBLIOGRAPHY: UNDERGROUND HYDROELECTRIC POWER PLANTS

J. Barry Cooke,* A.M. ASCE, and
Arthur G. Strassburger,** J.M. ASCE
(Proc. Paper 1350)

SYNOPSIS

This bibliography, 1912 through 1956, is based on a Report of the Power Division, Committee on Progress in Power Plant Design. The purpose of the report was to review existing literature and determine if there was need for ASCE sponsorship of a symposium on underground hydroelectric power plants. It was concluded that there is much general and specific literature available, but that a symposium is appropriate due to the many underground plants now in construction and planning. The symposium, scheduled for October 1957, will contain a paper on the first large underground plant in the United States, the Haas plant of The Pacific Gas and Electric Company.

Underground plants are being adopted to an increasing extent throughout the world, except in the United States. The question arises as to why they are not adopted in this country. It is concluded that very special conditions are necessary to make an underground hydro plant economic in the United States.

Preceding the bibliography is (1) a review of existing general literature, (2) a brief discussion, and (3) a list of the world's underground hydroelectric plants with some general data. The chronological bibliography includes brief resumes.

CONTENTS

Section	
A	Existing General Literature
B	Discussion

Note: Discussion open until January 1, 1958. Paper 1350 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, No. PO4, August, 1957.

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Section
C List of World's Underground Hydroelectric Plants with General Data and References
D Bibliography - Underground Hydroelectric Power Plants

Section A - Existing General Literature

The term existing general literature refers to literature on the general subject of the economics and design of underground power plants. Of the many articles in the Bibliography, two authors treat the general subject of "underground" versus "above ground" power plants. The majority of the articles discuss features of specific plants. The papers of two authors who have given general treatment to the subject will be briefly summarized.

1. Paper by J. D. Lewin, M. ASCE, Board of Water Supply Engineers of New York.

   DESIGN AND CONSTRUCTION OF UNDERGROUND HYDRO-ELECTRIC POWER PLANTS. Paper submitted at ASCE Annual Convention, Power Division, New York, Jan 1950. Not available in ASCE Library. Published versions:

   UNDERGROUND HYDRO-ELECTRIC POWER PLANTS. Engineering v 169 Feb 24 Mar 3, 10, 1950, p 225-26, 236-37, 264-65.

   UNDERGROUND HYDRO-ELECTRIC POWER PLANTS HAVE ECONOMIC AND OPERATIONAL ADVANTAGES, Civil Engineering v 20 Feb 1950 p 100-01.

   UNDERGROUND HYDRO PLANTS. Mechanical Engineering v 72 Apr 1950 p 323.

   UNDERGROUND HYDROELECTRIC PLANTS. Power Engineering v 54 Dec 1950 p 58.

   General paper on the status (1950) of underground plants and factors involved in economics and design of an underground development - stresses basic reason for going underground to be economics - describes the two basic schemes; the head-development (Swedish) and the tail-development (Swiss) - costs of Handeck and Innertkirchen - sharing of penstock pressure between rock and pipe shell improves underground economics - discusses arrangement of accessories, roof and wall design and treatment - economic benefits of unlined tunnels - various access arrangements - handling of ventilation.

2. Articles by Dr. Charles Jaeger, Consulting Civil Engineer, Water Turbine Department, English Electric Company Ltd., Rugby:

   a) UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, Civil Engineering and Public Works Review v 43 Dec 1948 p 620 v 44 Jan, Feb 1949 p 38, 85.

   Points out trend toward underground plants where special conditions make them economic and safer - summary of technical arguments in favor - detailed discussion of four hydraulic types of arrangements of the head tunnel, surge chamber and tailrace tunnel - figures of Innertkirchen, Pfaffensprung,

Erstfeld, Greina - Blenio scheme, Biaska - brief review of excavation and lined penstock design - bibliography.

b) UNDERGROUND HYDRO-ELECTRIC POWER STATIONS. Electrical Engineer and Merchandiser (Melbourne, Australia) v 27 June 15 1950 p 73-82.

c) A STUDY TOUR IN THE ALPS. Water Power v 3 May, June 1951 p 173-177, 213, 218.

Discussion of some general considerations on underground plants with specific data on the Santa Massenza II Project and the Giustine Project in the Italian Alps.

d) PRESENT TRENDS IN THE DESIGN OF PRESSURE TUNNELS AND SHAFTS FOR UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, (Paper No. 5978) Institution of Civil Engineers, Proceedings, v 4 pt 1 Mar 1955 p 116-200 (Abstracted in Water Power, v 7 Feb, Mar, Apr, May 1955, p 52, 94, 124, 162.

Since the majority of pressure shafts are connected to underground power houses, this paper contains much on underground plants.

A number of underground stations were built before the 1939-45 war, but most were on the continent of Europe. The postwar period has seen a tremendous development of this type of design in all parts of the world. Economic and financial advantages are at the root of the trend to more underground stations. Improved tunneling technique, larger stations, scenic requirements, safety, etc. are other factors. Types of underground plants listed and problems are discussed.

After discussion of hydrodynamics, the main body of the paper deals with pressure tunnel design. A table of existing pressure shafts gives physical data.

Underground power houses are analyzed from the point of view of overall dimensions, volume of excavation, design technique, and machinery. Sections of Innertkirchen, Santa Massenza, Santa Giustine, Isere-Arc, Peccia, Lavey, Clachan, Upper Morrison and Kemano are included. A tabulation of plants is given.

26 pages of discussion by others are included as well as comprehensive bibliography.

e) THE NEW TECHNIQUE OF UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, The English Electric Journal, v 14 No 2 June 1955 p 3-39.

Contains essentially the underground power house data of the above reference (d) with abbreviated treatment of pressure shafts. Tabulations are brought up to date.

Section B - Discussion

Before World War II the underground power plant was unusual and the few examples were primarily in Europe. Since the war underground schemes have been adopted at an increasing rate, not only in Europe, but all over the world. The exception has been in the United States. It is interesting that three of the world's first six plants, and none of the more than 160 plants

constructed since that time, have been in the United States. The world's first six underground plants are believed to be: Snoqualmie Falls, Wash. (1899); Fairfax Falls, Vermont (1904); Breckbergmuehle, Germany (1907); Mockfjard, Sweden (1910); Porjus, Sweden (1914); and P.G. & E.'s Spaulding No. 1, California (1917). The question arises as to why no underground plants have since been adopted in the United States, except the Haas plant which is currently under construction. The answer appears to be that in this country very special conditions are required to make the underground plant economic.

In nearly every case the reason for adopting an underground design has been basically economic. Many underground plants have cost less than the alternate surface scheme; or with credit for the additional effective head, they produce power at the lowest cost. Others are considered economic when credit is given to scenic benefits, freedom from slides, avalanches or freezing or other semi-tangible benefits. Improvements in tunnel driving methods, advances in design of pressure shafts and construction of large plants have improved the economics of going underground.

A factor that makes underground power plants more competitive in other parts of the world is the relatively higher cost of underground excavation in the United States, largely due to the higher labor cost. Another factor is that the underground development requires more labor and less material than the alternate surface development. Economics in other countries usually favor the use of relatively more labor and less material.

The general literature, reviewed in Section A, points out pros and cons regarding surface versus underground schemes. Alternate layout, costs, design considerations and other factors are all reviewed in sufficient detail to point out the economic possibilities of an underground plant. It is considered that more general literature is desirable to introduce the project planning engineer to the subject of underground schemes, and to interest him in considering such a scheme among the alternate layouts.

Underground plants have been sufficiently unusual that there is considerable detailed literature on the more than 120 existing developments. The economic study, design, construction and experience for many of these plants is well covered in the literature. These articles are very valuable in the technical information they contain and in their reminding the hydro-electric engineer that underground alternate schemes must not be overlooked.

Pacific Gas and Electric Company is constructing the first large underground plant in the United States, the Haas plant, and a paper on the plant is included in the 1957 Symposium. The Haas plant is underground for economic reasons. The factors of very high head (2450 feet), flat penstock profile and ideal massive granite combine to make the adopted scheme economic.

It is considered that a symposium on underground hydro-electric power plants together with the construction of the first major underground plant will stimulate the giving of full consideration to such layouts.

Section D - Bibliography for Underground Hydroelectric Power Plants

The bibliography is presented chronologically and in two parts. Part I covers the years 1912 through 1949 and has been taken from USBR Technical Bibliography No. 215, "Underground and Submerged Power Plants," by Ferd. Stenger. Part II covers the years 1950 through 1956 and contains brief abstracts of many of the articles. Part II is intended to be reasonably complete for literature in the English language but contains some foreign language articles. For references to specific plants, see Section C.

Part I - 1912 Through 1949

1912

1. FURTHER DEVELOPMENT AT SNOQUALMIE FALLS, M. T. Crawford. Engineering Record, v 65 Jan 13 1912 p 32.

2. PUGET SOUND TRACTION, LIGHT AND POWER COMPANIES SYSTEM, R. W. Van Norden. Journal of Electricity, v 28 Jun 1 1912 p 511.

1913

3. SUBTERRANEAN SWEDISH GENERATING PLANT. Electrical World v 61 May 10 1913 p 979. Description of hydraulic-electric generating and transmission equipment of the Vesterdalafoen Power Co. at Mackfjard, part of a 65,000 h.p. interconnected system.

1916

4. PACIFIC COAST HYDROELECTRIC DEVELOPMENT, J. Harisberger. Journal of Electricity, v 36 May 27 1916 p 403. Snoqualmie Power Plant.

1921

5. UNCAS PARTICULAR D'USINE HYDRO-ELECTRIC SOUTERAINE (AN UNDERGROUND HYDROELECTRIC PLANT), P. Basiaux. Revue Genéral de l'Electricité v 9 May 7 1921 p 656-57. Tunnel was constructed for diverting flow of water from Diege River, France - 200,000 h.p.

1923

6. WATER POWER DEVELOPMENT IN SWEDEN, W. Borgquist. Electrical World v 82 Oct 27 1923, p 851-55. Includes Porjus plant.

1928

7. UNDERGROUND POWER PLANTS (SULIE CENTRALI SOTTERANCE), G. E. Dolcetta. Energia Elettrica v 5 Dec 1928 p 1407. Features of Alto Flumendosa hydro developments having two plants in rock 100 feet deep.

1931

8. HYDRO PLANT BUILT ABOVE THE ARCTIC CIRCLE, G. Willock. Power v 73 Jun 23 1931 p 986. Porjus plant.

1932

9. LES INSTALLATIONS HYDRO-ELECTRIQUES DE LA TRUYÈRE (THE HYDRO-ELECTRIC INSTALLATIONS ON THE TRUYERE RIVER), J. Dumas. Génie Civil (Paris), v 101 Sep 17 1932 p 269.

1935

10. PRINCIPALI IMPIANTI IDROELETTRICI EUROPEI CON CENTRALE SOTTERRANEA E LORO PARTICOLARITA COSTRUTTIVE (PRINCIPAL EUROPEAN HYDRO-ELECTRIC PLANTS WITH UNDERGROUND POWER HOUSES AND THEIR CONSTRUCTION FEATURES), R. Mosca. Energia Elettrica v 12 Feb 1935 p 85.

1938

11. KRANGEDE HYDRO-ELECTRIC POWER SCHEME. Engineer v 166 Aug 19, 26, Sep 2, 1938 p 193, 218, 244. 40 meters below surface.

1941

12. UNDERGROUND BOMBPROOF HYDRAULIC GENERATION STATION. Electrical News and Engineering v 50 May 1 1941 p 27.

1942

13. DAS KRAFTWERK INNERTKIRCHEN, A. Kaech, H. Julliard, and F. Aemmer. Schweiz Bauzeitung v 120 Jul 18, 25, Aug 1, 8, Oct 31, 1942 p 25, 36, 47, 61, 208. Supplemental power plant equipment description.

14. DAS KRAFTWERK INNERTKIRCHEN, DIE ZWEITE STUFE DER KRAFTWERKE OBERHASLI, J. Leuenberger and H. Ludwig. Wasser- und Energiewirtschaft v 34 Jul-Aug 1942 p 66.

15. DER ZWEITE AUSBAU DER OBERHASLI-KRAFTWERKE. Assn Suisse des Electriciens Bul v 33 Oct 7 1942 p 529-42, see also: Bul Technique de la Suisse Romande v 68 Nov 14 1942 p 265-71, Nov 28 p 277-84; Engineer v 175 Mar 5 1943 p 187-89, Mar 12 p 208-11, Mar 19 p 226-29; Mar 26 p 244-47, Apr 2, p 264-67, Apr 9 p 286-89, Apr 16. Second extension of Oberhasli hydroelectric powerplants; illus. description of reservoir, pressure tunnel, underground powerplant of Innertkirchen and its equipment and subaqueous canal.

1943

16. BOMB-PROOF HYDRO PLANT. Elec Times v 103 Mar 4 1943 p 240-41. General layout and equipment of Innertkirchen, Switzerland, plant.

17. OBERHASLI HYDRO-ELECTRIC POWER SCHEME. The Engineer v 175 Part II Mar 12 1943 p 208-11, Part V Apr 2 p 264-7, Part VI Apr 9 p 286-9, Part VII Apr 16 p 305-9. Part II, history, general outline, preliminary construction of Innertkirchen power plant - Part V, main power station - unreinforced concrete roof lining - tailrace parallel to long. axis of turbines - generators mounted directly on turbines - construction progress - Part VI, illustrations of construction - erection of valves, turbines and generators - Part VII, sectional model of plant - transformers, ventilating equipment, switchgear, etc.

1944

18. DER BAU DER WASSERKRAFTANLAGE INNERTKIRCHEN, M. Tzchaetzch. VDI Zeitschr v 88 Aug 5 1944 p 421-29.

1946

19. MULLARDOCH-FASNAKYLE-AFFRIC HYDRO-ELECTRIC SCHEME, Engineer v 182 Sep 27 1946 p 284 - Scheme involves dam 120-ft. high at Loch Mullardoch, tunnel diverting its waters to Loch Benevenn through underground power house.

1947

20. POWER SUPPLY IN SWEDEN, Swedish State Power Board 1947 p 1-32. Sweden's complete power system, municipal and private, steam and hydroelectric, physical data - bits of data on Krangede, Järpstrommen, and Porjus underground hydro plants.

21. WATER POWER: CORNERSTONE OF SWEDEN'S INDUSTRIAL DEVELOPMENT, H. G. Tonndorf. Power v 91 Mar 1947 p 168-170.

22. HYDRO IN WARTIME GERMANY. ASME-Transactions, A. Hoefle, v 69 Jul 1947 p 541-45.

23. UNDERWATER POWER PLANTS. Elec. Times v 111 n 2902 Jun 19 1947 p 707-08. Review of Fischer's hydroelectric schemes in Germany.

1948

24. GRANDE DIXENCE-DONNEES TECHNIQUES DU PROJET EOS, L. Favrat, A. Livic. Wasser und Energiewirtschaft v 40 Jan 1948 p 1-6. Four power-plants, three are underground.

25. UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, C. Jaeger. Civil Engineering and Public Works Review (London) v 43 Dec 1948 p 620-23, v 44 Jan, Feb 1949, p 38, 85. Review of Underground stations in Sweden, Switzerland, Scotland, and Australia; diagrams of four different types of underground stations; technical drawings of plant at Innertkirchen, Pfaffensprung and Erstfeld, all in Switzerland - Bibliography.

26. L'USINE HYDRO-ÉLECTRIQUE DE LAVEY (THE LAVEY HYDRO-ELECTRIC DEVELOPMENT), P. Meystre. Wasser und Energiewirtschaft v 40 Jul-Aug 1948 p 86.

1949

27. UN GRANDE PROJET POUR L'UTILISATION DES RESSOURCES HYDRAULIQUES AUSTRALIENNES, L. Barker. Houille Blanche, v 4 Jan-Feb 1949 p 9.

28. KIEWA HYDROELECTRIC PROJECT: FIVE GENERATION STATIONS TO BE UNDERGROUND, N. O. Hall. Compressed Air Magazine v 54 Feb 1949 p 26.

29. KIEWA HYDRO-ELECTRIC SCHEME, AUSTRALIA 289,000 KW PROJECT. Water and Water Engineering, v 52, Mar 1949 p 112.

30. AUSTRALIAN POWER PLANTS BEING BUILT UNDERGROUND, H. Fletcher. Industry and Power v 56 Apr 1949 p 91.

31. LA VENTILAZIONE E L'ILLUMINAZIONE DELLE CENTRALI IN CAVERNA (VENTILATION AND ILLUMINATION OF UNDERGROUND POWERHOUSES), C. Ritter. L'Energie Elettrica v 26, May 1949.

32. SNOWY RIVER. Water Power v 1 May-Jun 1949 p 101.

33. TUNNELLING PRACTICE. Water Power v 1 Jul-Aug 1949 p 172. Review of problems of excavating in rock with special reference to construction of generating stations underground.

34. HYDROELECTRIC DEVELOPMENTS IN SOUTH-EASTERN AUSTRALIA. Engineer v 187 Jun 10 1949 p 628.

35. HYDRO-ELECTRIC DEVELOPMENTS IN ITALY. Engineering v 168 Sep 30 1949 p 342.

36. LAVEY HYDRO-ELECTRIC STATION OF LAUSANNE CITY ELECTRICITY SUPPLY, M. Ebersberger. Brown Boveri Review v 36 Oct.-Nov 1949 p 330-47. Alternators, main transformers, switchgear and protective and regulating equipment in 100,000 H.P. underground plant - geological conditions deciding factor in adopting underground scheme.

37. UNDERGROUND POWERHOUSES, A. L. Alin. U. S. Army Corps of Engrs., Office of the District Engineer, Omaha, Nebraska, Nov 1949.

Part II - 1950 Through 1956

1950

38. SPANISH WATER POWER DEVELOPMENTS, Mr. Rolt Hammond. Water Power v 2 Jan-Feb 1950 p 44-48. Includes very short discussion of Tranco de Beas Project - two units at 27,000 hp - transformers outside - BV inside power house.

39. UNDERGROUND HYDRO-ELECTRIC POWER PLANTS, J. B. Lewin. Engineering v 169 Feb 24 1950 p 226, Mar 3 p 236-37, Mar 10 p 264-65. See Section B for resume.

40. DAS KRAFTWERK CALANCASCA. Wasser und Energiewirtschaft v 42 Mar 1950 p 33-39, Apr 1950 p 57-60. Review of Calancasca power development in Switzerland underground station with impulse turbine 1300 ft head - illustrations.

41. KRAFTWERKBAUTEN IN NORDITALIEN. G. A. Toendury. Schweiz Bauzeitung v 68 Mar 11 1950 p 121-26, Mar 18 p 133-35, Apr 1 p 170-73, Apr 8 p 184-89. A review of power output - 73% supplied by hydroelectric plants. Hydroelectric plants are mostly underground.

42. THE HARSPRANGET HYDROELECTRIC POWER PLANT, SWEDEN. Swedish State Power Board, Apr. 1950.

43. POWER STATION OF RADICAL DESIGN PROPOSED FOR EL SALVADOR, K. E. Sorensen. Engineering News-Record v 144 June 15 1950 p 44-45. Description of proposed Guayabo underground plant on Lempe River. Power plant directly below dam - vertical penstocks, horizontal shaft units - two tail tunnels - advantages of design.

44. UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, C. Jaeger. Electrical Engineer & Merchandiser v 27 Jun 15 1950 p 73-82. See also Section A of this paper.

45. MODEL TESTS ON CLACHAN UNDERGROUND POWER STATION, G.F.W. Adler. English Electric Journal v 11 Jun 1950 p 119-27.

46. DER WEITERAUSBAU DER WASSERKRAEFTI IM OBERHASLI, H. Ludwig. Wasser und Energiewirtschaft v 42 Jun-Jul 1950 p 88-91, Aug p 127-29, Sep p 166-70. Water power in Oberhasli, Switzerland - illustrated description of plants under construction - Handeck II and Oberaar plants.

47. HYDRO PLANTS IN CAVES (Ventilation and Illumination Aspects). Electric Light and Power v 28 Jul 1950 p 77-78. Abstract of article by C. Ritter, "L'Energia Elettrica, Italy." Data applies to the power plants in caves of the Societa Adriatica di Elettricita in Italy.

48. AN UNDERGROUND STATION, Carlo Semenza. Water Power v 2 Jul-Aug 1950 p 144-51. Principal features of the Lumiei, Italy hydro-electric project - most of article devoted to high concrete arch dam - small portion devoted to underground power station. Butterfly valve located in chamber with tunnel access - transformers and switchgear underground - due to poor rock conditions, walls and concrete pillars were constructed with reinforced buttresses and intermediate relieving arches.

49. UNDERGROUND POWER STATION: FEATURES OF THE LAVEY UNDERTAKING. Electrical Review v 147 Nov 3 1950 p 705-___.

50. UNDERGROUND POWER STATIONS, Luigi Gallioli. Water Power v 2 Nov-Dec 1950 p 227-36, 246. Factors to be considered in underground schemes Stazzona project, Milan is typical underground scheme - plant placed underground for security reasons - transformers, switchboards, and auxiliary equipment in two lateral tunnels - tunnel completely concrete lined, power station housed inside an inner structure which is independent of the outer - cranes carried by reinforced pillars - curved walls used to close the two ends of the outer nave-curved walls were used to transmit the thrust of the rock to the walls - to avoid transmission of vibrations - machines mounted on independent isolated bases - located in very poor formation.

51. UNDERGROUND HYDROELECTRIC PLANTS. Power Engineering v 54 Dec 1950 p 58-59, 116-20. Summarizes accomplishments in underground plants to date - great differences in plants, as to location, size, type, reasons for building underground, etc. - briefly discusses layout details.

52. DESIGNING AND DIMENSIONING OF THE HYDRO-ELECTRIC POWER PLANTS OF THE SWEDISH STATE BOARD, K. J. P. Wittrock and K. G. G. Pira, Swedish State Power Board Publication No. 8 1950.

53. UNDERGROUND HYDRO-ELECTRIC POWER STATIONS IN SWEDEN, A. Rusck and G. Westerberg, Swedish State Power Board Publication No. 9 1950.

54. NORWEGIAN HYDRO-ELECTRIC POWER STATIONS BUILT INTO ROCK, Ragar Heggstad. Fourth World Power Conference 1950 v 4 Section H/Z Paper 1 p 2050.

55. SWEDISH PRACTICE IN WATER POWER DEVELOPMENT, D. Westerberg and B. Hellstrom. Fourth World Power Conference 1950 v 4 Section H/I Paper 2 p 2021.

56. UNDERGROUND HYDRO-ELECTRIC POWER STATIONS IN SWEDEN, A. Rusck and G. Westerberg, AIEE Transactions v 69 1950 p 928-32. Factors to consider in developing underground plants.

57. UNDERGROUND HYDRO-ELECTRIC POWER PLANTS, Paul E. Gisiger, AIEE Transactions v 69 1950 p 1169-75. Lists factors to consider in developing underground plants - reasons for going underground - discusses layout details of various plants, giving advantages and disadvantages of different schemes.

1951

58. UNUSUAL ROCK-FILL DAM BUILT FOR POWER IN SWEDEN. Engineering News-Record v 146 Jan 18 1951 p 47-48. Describes and illustrates Harspranget Dam with brief description of power plant.

59. HARSPRANGET AMBITIOUS SWEDISH UNDERTAKING, F. Illingworth. Mine & Quarry Engineering v 17 Jan 1951 p 11-13. Excavation of Harspranget power station - 225 ft down in granite.

60. A NEW SWISS LOW-HEAD STATION. Water Power v 3 Feb 1951 p 45-53. Lavey power station, Switzerland - 246' long steel lined penstock shaft - underground because of alluvial ground - generators are closed circuit air cooled with water cooled radiators - surface transformers, switchgear and switchhouse - two 60-ton cranes.

61. THE LAVEY HYDRO-ELECTRIC POWER STATION FOR LAUSANNE. Engineering v 171 Mar 23 1951 p 333-36. Complete and detailed description of electrical equipment of Lavey plant in Switzerland - steel lined penstocks and trapezoidal tailrace channel - only bearing and exciter above main floor to minimize headroom - all power house switchgear air operated to minimize fire hazard - transformers and main switchgear on surface.

62. GUAYABO HYDRO-ELECTRIC PROJECT. Water Power v 3 Mar 1951 p 99-101. In El Salvador - power house directly below intakes - horizontal Francis turbines to eliminate penstock and draft tube bends - single tailrace tunnel - elevator or inclined tunnel access - low tension cables to transformers above ground.

63. A STUDY TOUR IN THE ALPS. Charles Jaeger, Water Power v 3 Jun 1951 p 213-18. See also Section B of this paper. Italian Edison rule on transformers, above ground if cable is less than 820 feet, under if greater than 1150 feet - Santa Massenza II Project - penstock slope 1 on 0.85 - Santa Guistina - heavy reinforced struts in turbine floor to resist plastic movement of rock - power cables in access tunnel.

64. L'AME'NAGEMENT DE LA CHUTE DE PEYRAT-LE-CHATEAU, G. Rodriquez. Génie Civil v 128 Jun 1 1951 p 210-212. Utilization of head of Peyrat-Le-Chateau, Haute Vienne, France. 830 ft head underground station.

65. HYDRO-ELECTRIC SCHEMES: MODERN TRENDS IN CIVIL ENGINEERING, T. A. L. Paton, Engineering v 172 Aug 24, 1951 p 253-54. Modern trends in design and construction are discussed including dams, tunnels, penstock and powerhouses- pros and cons on underground plants.

66. HJALTA POWER STATION, I. Nilsson - Stig ASEA Journal v 24 Aug-Sep 1951 p 111-133. Complete issue dealing with this Swedish power plant - most of article devoted to electrical equipment with a few comments on physical layout. Three vertical penstocks - tailrace tunnel 1450 sq. ft. in area, 4 miles long - transformers, 380 v switchgear and duplicate control board underground - transformers in concrete bays - busbars in vertical shaft - access shaft has two passenger elevators, stairs and a clear space for lowering equipment - machine hall has false room - crane supported on independent columns tied to rock slides - curtain walls.

67. MONTE ARGENTO HYDROELECTRIC SCHEME, ITALY. The Engineer v 192 Sep 14 1951 p 346-7. General outline of scheme - Monte Argento and Cotilia underground power stations - general data for Monte Argento - gate controlled - transformers and switchgear above ground - underground tail surge chamber.

68. BRAZIL'S PARAIBA - PIRAI DIVERSION PROJECT, A. J. Ackerman, F. T. Mathias, G. O. Vogan. Civil Engineering v 21 Nov 1951 p 652-56. Forcacava - steel lined shaft penstock with liner taking 30% of water pressure.

69. WORLD'S BIGGEST ALUMINUM PLANT. Engineering News-Record v 147 Nov 29 1951 p 40, 43. General outline of Alcan Project. Brief outline of power plant, turbines, generators.

70. ICELAND BUILDS NEW HYDRO-ELECTRIC PLANTS UNDERGROUND. Power Engineering v 55 Nov 1951 p 87-89. Brief general resume of Irofoss plant, Sog Development - project profile and general sections of power house.

71. FIRST UNDERGROUND POWER PLANT? Engineering News-Record v 147 Dec 6 1951 p 38. Snoqualmie Falls, Washington plant completed 1899 - placed underground to avoid freezing spray of the falls - steel penstocks 7-1/2 feet dia. carried down shaft also containing elevator and conductors - four 2000 hp turbines. Another plant built 1904 at Fairfax Falls, Vermont - two 350 kw generators - flood in 1927 broke through shallow rock roof, destroying plant.

72. LES TURBINES DE LA CENTRALE DE SALANFE - MIEVILLE, P. Pinguod. Bulletin Technique de la Suisse Romande v 77 Dec 15 1951 p 361-65. Salanfe-Mieville hydroelectric power plant in Switzerland - hydraulic data.

73. COORDINATION OF THE DEVELOPMENT OF INDUSTRIES AND THE DEVELOPMENT OF POWER RESOURCES IN NORWAY, The Federation of Norwegian Industry, Transactions of the Sectional Meeting, World Power Conference 1951 v III p 432. Very brief incidental data on the Maar, Tyin, and Vinstra underground hydroelectric plants.

1952

74. ARCTIC POWER, J. H. M. Sykes. Water Power v 4 Jan 1952 p 4-14. Complete proposed development of Stora Lule River in Sweden - Harspranget power station, the first to be constructed is briefly discussed - mention made of Kilforsen and Lasele projects, both future underground plants.

75. LONG PRESSURE TUNNEL FEATURES ITALIAN PLANT. Engineering News-Record v 148 Mar 20 1952 p 72-73. News article 5.7 mile long pressure tunnel serves 50,000 kva underground generating plant at Ala.

76. HARSPRANGET HYDRO-ELECTRIC SCHEME, SWEDEN. The Engineer v 193 Mar 28 1952 p 431-2. Scheme typical of Swedish developments with rock fill dam, intake works to underground power station below it, unlined tailrace tunnel of large cross sectional area - illustrations.

77. LAVEY HYDRO-ELECTRIC STATION, Abstracted from the "Bulletin Technique de la Suisse Romande." Water Power v 4 Mar 1952 p 117-120. Brief and general outline of civil engineering features of Lavey power station in Switzerland - illustrations - cost data.

78. SOVERZENE GENERATING STATION, Dr. Ing. Mario Mainardis, Water Power v 4 Mar 1952 p 95-100, Apr 124-29. Describes chief features of Soverzene underground power plant - plan chosen to utilize maximum head - length of penstock reduced to 1/3 that required for surface scheme - prestressed concrete pipe used in lieu of welded plate - precast sections were 14'9" long, 8'4-1/2" in diameter and 7-1/2" thick - concrete poured into steel forms, outer of which remained in place as watertight membrane - then prestressed by wrapping with high tensile wire - placed and butt-welded at edges of liner - outside gunited and encased in concrete - transformers in separate chamber - two independent fire systems,

carbon dioxide, and atomized water - fresh air brought in through false arch over entrance tunnel - to avoid flooding in case of burst in penstocks or turbine casing, brick walls between turbine compartments and tailrace were built weak enough to give way under very slight head - watertight door between turbine compartments.

79. THE GLEN AFFRIC SCHEME. Water Power v 4 May 1952 p 185-91, Jun p 224-32, Jul p 270-74. Glen Affric Scheme of North Scotland - two dams, two tunnels, a main generating station and subsidiary underground station at Mullardoch - Mullardoch described briefly in second part.

80. THE 350-MW STATION OF THE SWEDISH STATE POWER BOARD AT HARSPRANGET. Engineering v 173 Jun 20 1952 p 783. Description - rockfill dam from rock excavated from power station - 1-3/4 mile tailrace tunnel of 2045 sq. ft. cross section - transformers underground - 380 KV cables in shaft to surface switchgear.

81. ALCAN'S NECHAKO - KEMANO - KITIMAT PROJECT, H. R. Rice. Canadian Mining Journal v 73 Jun 1952 p 79-87, Jul p 63-70. Rock elasticity investigation - powerhouse construction - general outline of complete project - Part II gives descriptive account of rock and tunnel excavation work - drilling and blasting procedures used in power chamber.

82. KEMANO, F. H. Stephens. Western Miner v 25 Jun 1952 p 55-62. Mining work necessary for construction of hydroelectric power plant at Kemano - construction of power house 1400 ft. underground - driving of raises and tunnels - characteristics of transmission line, equipment, and transportation.

83. LES CENTRALES HYDROELECTRIQUES SOUTERRAINES (UNDERGROUND HYDRO-ELECTRIC POWER STATIONS), Mario Mainardis. Monde Souterraine v 17 Jun 1952 p 730---.

84. SANTA GIUSTINA. Water Power v 4 Aug 1952 p 289-98, Sep p 324-33. Outstanding features of hydro-electric scheme on River Noce in Northern Italy - first part primarily devoted to Santa Giustina dam and pressure tunnel - second part to the underground power plant - well illustrated - surge chamber comprised of two parts, vertical shaft and helicoidal shaft which ascends with increasing inclination and variable horizontal area - one of very few underground plants where power plant rotary valves are located in power chamber adjacent to turbines - transformers and switchgear are on surface - vertical shaft contains busbars, air ducts, stairway, and elevator - power chamber fully lined and has false ceiling - heavy duty access by inclined tunnel - tailrace tunnel lined.

85. NORTH OF SCOTLAND HYDRO-ELECTRIC BOARD. Engineering v 174 Sep 12 1952 p 343. Short report on redesigning of Glen Morison hydro-electric scheme to include underground power house - station under Dundreggan dam.

86. SETTING UP SHOP IN THE WILDERNESS FOR HUGE ALUMINUM HYDRO PROJECT, L. L. Wise Engineering News-Record v 149 Sep 25 1952 p 32-37. Establishing of plant for Kemano - Kitimat project.

87. NECHAKO - KEMANO - KITIMAT HYDRO-ELECTRIC POWER DEVELOPMENT AND ALUMINUM REDUCTION PLANT, F. L. Lawton and J. S. Kendrick. Engineering Journal v 35 Sep 1952 p 915-929.

88. THE GLEN AFFRIC SCHEME OF THE NORTH OF SCOTLAND HYDRO-ELECTRIC BOARD. Engineering v 174 Oct 10 1952 p 457-60. Full description of development including two dams and two reservoirs with connecting tunnel, plus another dam and Mullardoch power station - to utilize drop in head between the two reservoirs, small subsidiary underground powerhouse planned - 12' dia. access shaft - turbine will discharge through vertical draft tube into tunnel which rejoins main tunnel between two reservoirs - plant placed underground partly because of scenic reason - illustrations - first part describes Mullardoch.

89. TROLLHÄTTAN. Water Power v 4 Oct 1952 p 364-70, 387. Hydro-electric installations at Tröllhattan on Göta River in Sweden - one of new plants is Hojum underground station - described briefly - arched roof concreted with crane supported on columns. Walls are exposed rock - heat from transformers circulated through power house to heat chamber and keep walls dry - water cooled transformers located in separate underground cubicles - switching outdoors.

90. ERFAHRUNGEN BEIM BAU DES KRAFTWERKES HANDECK II (EXPERIENCE GAINED DURING THE CONSTRUCTION OF HENDECK II POWER PLANT), J. Bächtold. Schweitz Bauzeitung v 70 Oct 4 1952 p 573-77, Oct 11 p 587-90, Oct 25 p 612-14.

91. WORLD'S LARGEST UNDERGROUND POWER PLANT, L. L. Wise. Engineering News-Record v 149 Nov 13, 1952 p 31-36. Illustrated discussion of powerhouse, penstock, and power tunnel excavation for Kitimat - Kemano scheme. Lists reasons for going underground.

92. 350 MW HYDRO-ELECTRIC GENERATING STATION AT HARSPRANGET, SWEDEN. Engineering v 174 Nov 14 1952 p 617-19. Article in four parts giving complete and detailed description of Harspranget development - second installment devoted to power station - upper portion of four vertical penstocks reinforced concrete designed to take external pressure - rock carries internal pressure - lower portion steel lined - separate generator room and transformer cubicles - roof of sound granite strengthened by one 1" dia. bar per 11 sq. ft. of roof, grouted to depth of 10 to 16.5 ft. and covered with reinforced gunite - drainage provided - provision made for false ceiling over generator room if necessary - 350 kv leads from transformers in separate shaft - control room underground - turbines discharge into surge basin thence into 2045 sq. ft. discharge tunnel 1.8 mi. long. Article notes all Swedish underground plants are underground for economic reasons, with safety being additional but not decisive factor.

93. DER AUSBAU DER WASSERKRÄFTE IM OBERHASLI (THE DEVELOPMENT OF THE OBERHASLI POWER SCHEME), H. Julliard. Schweitz Bauzeitung v 70 Nov 29 1952 p 677-82. (5th of 7 parts).

94. ALCAN-BRITISH COLUMBIA POWER PROJECT UNDER CONSTRUCTION, W. G. Huber. Civil Engineering v 22 Nov 1952 p 938-43. 8 units first with 16 ultimate of Kemano - access tunnel 1450' long - reinforced concrete arch roof - concrete block curtain walls - 138-287 kv transformers in vaults in main chamber - 2200' of single-core 301 kv oil-filled cable.

95. FURTHER HYDRO-ELECTRIC POWER DEVELOPMENTS IN SCOTLAND,

J. C. Beverly. English Electric Journal v 12 Dec 1952 p 5-22. Mullardoch-Fasnakyle-Affric Scheme, including Mullardoch underground power station.

96. UNDERGROUND POWER STATIONS, W. Weissel. Institution Engineers (India) Journal v 33 Dec 1952 p 195-208. Advantages and disadvantages - factors in design - data on plants in Sweden, Norway, Germany, Russia, France, Spain, Austria, Switzerland, Italy and Australia.

97. KEMANO - KITIMAT, PRODUCER OF PACKAGED HYDRO POWER, F. L. Lawton. American Power Conference Proceedings v 14 1952 p 165-176.

98. LES USINES HYDRO-ELECTRIQUES SOUTERRAINES EN SUEDE, Rusck and Westerberg. Le Monde Souterrain v 69-70 1952.

99. THE PRESENT STATE OF UNDERGROUND PENSTOCK TECHNIQUE, Joseph Talobre, La Houille Blanche v 7 1952 p 513-531. (Translation by Jan C. Van Tienhoven). Comprehensive article on design of pressure shafts - important to underground plants because of economics involved in pressure shaft design.

1953

100. NECHAKO - KITIMAT WATER PROJECT IN BRITISH COLUMBIA, F. L. Lawton. Water and Water Engineer v 57 Jan 1953 p 3-11.

101. VAST HYDRO SCHEME BECKONS U.S. CONTRACTORS, W. A. Dexheimer. Engineering News-Record v 150 Feb 5 1953 p 58-66. Complete outline of Australia's Snowy Mountains Hydro-electric scheme - underground power plant on Tumut River (T-1) ready for bids early in 1952.

102. TUNNELS AND UNDERGROUND PENSTOCKS REQUIRE A MILLION CUBIC YARDS ROCK EXCAVATION (ALCAN - BRITISH COLUMBIA HYDRO PROJECT), W. G. Huber, Civil Engineering v 23 Feb 1953 p 102-107. Two 11' dia. penstocks in concrete backfilled shaft - penstock slope 48° from horizontal - 4466' long - steel plate up to 1-15/16" thick - 1 penstock to 4 units.

103. NEUE MITTEL - UND HOCHDRUCK-WASSERKRAFTANLAGEN (NEW MEDIUM - AND HIGH-PRESSURE WATER POWER SCHEMES), F. Tölke. VDI Zeit v 95 March 11 1953 p 225-32.

104. UNDERGROUND HYDRO PLANT BOOSTS RIO POWER, E. A. Johanson. Electrical World v 139 Mar 23 1953 p 130-33. Forcacava (now Nilo Pecanha) - 541 ft. horizontal access tunnel - 2 - 125 ton cranes - engineering details - illumination and ventilation - plans.

105. BIG UNDERGROUND POWER PLANT PUT IN SERVICE IN SWEDEN. Engineering News-Record v 150 Mar 26 1953 p 77.

106. DEVELOPMENT OF WATER RESOURCES IN FRENCH NORTH AFRICA, R. Butler. Water & Water Engineering v 57 Mar 1953 p 91--. Recent accomplishments by French Government on construction for water supply, irrigation and hydroelectric power - includes 6 mile tunnel to underground plant, biggest plant in Algeria.

107. PHILIPPINES PUSH THEIR HYDRO PROGRAM. Engineering News-Record v 150 Apr 16 1953 p 43-47. Two hydro-electric developments in

Philippines - Ambuklao project in Luzon is biggest - underground plant to house three generators - 39,500 hp at 572 ft. head - tail tunnel 1-1/2 miles long - two sloping access tunnels - machine hall radial in plan.

108. AT KEMANO, ALCAN COMES UP WITH ANOTHER "BIGGEST." Western Construction, v 28 Apr 1953 p 102 - Briefly discusses impulse turbines to be used at Kemano - size compared to other large units.

109. ELECTRICAL ENGINEERING AND OPERATING ASPECTS OF ALCAN'S B.C. PROJECT, J. T. Madill. The Engineering Journal, v 36 Apr 1953 p 363-74.

110. ENGINEERING CONSIDERATIONS OF AMBUKLAO HYDROELECTRIC PROJECT IN AGNO RIVER, LUZON, A. P. Buenaventura. Philippine Engineering Record v 14 Apr 1953 p 6-11. Overall development of Agno river, with main dam of rock fill type, and underground power plant.

111. THE KITIMAT HYDRO-ELECTRIC POWER DEVELOPMENT SCHEME, F. L. Lawton. Engineers Digest v 14 Apr 1953 p 113-14, 148. General but complete outline of Alcan scheme - detailed information on power plant electrical equipment.

112. SNOWY MOUNTAINS. Water Power v 5 Apr 1953 p 131-139, May p 164-72, Jun 204-12. Snowy Mountain project of Australia - Part One, history of project - Part Two, technical and economic background - illustrates proposed layout of 8 dams, 85-1/2 miles of tunnels and 16 power stations - many will be underground - Part Three illustrates proposed underground power station T-1 - vertical penstocks with turbine valve in power chamber adjacent to turbines - control room at one end of generator hall - transformers underground in isolated hall - access by inclined tunnel - tailrace tunnel 21 ft. dia., lined, 3600 ft. long.

113. THE NECHAKO - KEMANO HYDRO-ELECTRIC SCHEME IN BRITISH COLUMBIA. The Engineer v 195 May 29 1953 p 777-79, Jun 5 p 811-12. General review of entire scheme.

114. VOM BAU DES KRAFTWERKS OBERAAR (CONSTRUCTION OF OBERAAR POWER PLANT), J. Bächtold. Schweitz Bauzeitung v 71 May 1953 p 271-7. Cableway - concreting plant - concrete conduits - gamma ray analysis of steel plate for pressure shaft - illustrations.

115. COMPLEX EXCAVATION PATTERN CUTS OUT UNDERGROUND POWERHOUSE, W. G. Huber. Civil Engineering v 23 Jun 1953 p 396-401. Kemano - sixteen 140,000 HP units - 800' rock overburden over crown. 1550' tailrace tunnel and 1340' access tunnel both 27' x 27' - concrete arch machine hall roof - columns and girts anchored to wall rock - two 225-ton cranes of 50' span - transformers in machine hall - oil-filled cables to sw. yd. - fresh air enters through access tunnel - water supply from tailrace - communication lines, controls, etc. in access tunnel floor.

116. ISÈRE - ARC DEVELOPMENT, C. Jaeger. Water Power v 5 Jul 1953 p 256-62, Aug p 301-04. One of projects of Electricité de France - special reference to design of the dam, surge chamber and pressure shaft - power plant data given.

117. THE PARAÍBA-PARAÍ DIVERSION. Water Power v 5 Aug 1953 p 287-93. Diversion scheme to obtain additional power for Sao Paulo, Brazil -

Forcacava underground station situated below and adjacent to existing surface Fontes power station - more economical to excavate new underground station than to attempt to enlarge Fontes, situated in a narrow valley.

118. UEBER DEN STAND DER ARBEITEN BEI DER ERSTEN BAUETAPPE DER MAGGIA-KRAFTWERKE. Schweiz Bauzeitung v 71 Sep 26 1953 p 562-69. Construction work on Maggia power plants - Peccia - Cavergno - Verbano - particulars of dams and pressure tunnels - illustrations.

119. KONAR DAM NEARING COMPLETION, W. Weissel. Indian Journal of Power & River Valley Development v III Sep 1953 p 1-4. Review of Konar Dam in Damodar Valley - Konar underground power plant directly beneath dam.

120. SPECIAL FEATURES OF THE AFFRIC HYDRO-ELECTRIC SCHEME (SCOTLAND), C.M. Roberts. Institution of Civil Engineers Proceedings v 2 Pt. I Sep 1953 p 520---.

121. UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, K. Seetharamiah. Indian Journal of Power & River Valley Development v III, Sep 1953 p 5-13, 30. Discussion of various types of underground power stations, advantages and disadvantages - history of underground station development - plants operating under head from 183 to 2235 ft. - claims first plant built in Switzerland in 1897 - modern tunneling techniques chiefly responsible for development - various lining schemes - trend to place transformers underground - Kagginfon station in Norway has transformers in access tunnel - four types of tailrace - Brommat, France; Innertkirchen, Switzerland; and Santa Massena, Italy are briefed - underground station being considered for Koyna Project in India.

122. USINA SUBTERRANEA DE CUBATAO, B. F. Barros Barreto. Engenharia v 12 Sep 1953 p 3-12. Underground hydroelectric power station of Cubatao, Brazil.

123. THE FORCACAVA HYDRO-ELECTRIC SCHEME, L. v. Rabcewicz. Water Power v 5 Sep 1953 p 333-37, Oct p 370-77, Nov p 429-35. Excavation for underground machine hall with adjoining pressure shaft, access and discharge tunnels for power plant at Forcacava, Brazil - working methods adopted in unstable rock - placed underground for several reasons; saving of foreign currency because of reduced amount of imported steel; inadequate space for surface building and switchyard; and risk of landslides to surface building - penstock steel lining based on assumption that 70 percent of pressure be borne by rock - power chamber fully concrete lined - broken rock caused overbreaking - inclined slip planes encountered - concrete columns erected during excavation heavily anchored to rock and concrete struts required to hold some columns - transformers in machine hall.

124. L'USINE HYDROELECTRIQUE D'HARSPRANGET, SUR LE FLEUVE LULEALV. Génie Civil v 130 Oct 1 1953 p 361-4.

125. ELECTRICAL ASPECTS OF ALCAN'S KEMANO - KITIMAT HYDRO-ELECTRIC POWER DEVELOPMENT, F. L. Lawton, AIEE, Transactions v 72 Pt III Oct 1953 p 1019 - (Also abstracted in Electrical Engineering

v 73 Feb 1954 p 124) - Thorough presentation of electrical mechanical features of Kemano power house and transmission - tabulations of turbine, generator and transformer data - ventilation.

126. SALANFE DEVELOPMENT. Water Power v 5 Oct 1953 p 378-85, 392. Mieville, Switzerland, plant special alloy penstock of 2-inch plate - ventilation includes dryer - transformers outdoors.

127. TUNNEL AND POWERHOUSE EXCAVATION AT KEMANO, B.C. FOR ALCAN HYDRO POWER, F. T. Matthias, C. W. Abrahamson. Canadian Mining and Metallurgical Bulletin v 46 Oct 1953 p 603-21. Excavation and driving methods in construction of 10 mile 25 ft. horseshoe tunnel, two penstocks, powerhouse cavern, and access tunnels - geology of region - mechanized equipment and performance.

128. HARD HATS VS. THE WILDERNESS ON THE ALCAN PROJECT. Western Construction v 28 Nov 1953 p 66-71, Dec p 64-67, 81. Outlines project - construction methods used in excavating for the powerhouse, penstocks, intake and tunnel.

129. WORLD'S TALLEST DAM RISES IN SWITZERLAND, Waldo G. Bowman. Engineering News-Record v 151 Dec 3 1953 p 32-41. Grand Dixence and Mauvoisin Projects will include two world's highest dams - projects coordinate to supply three power plants - construction plans and procedure for Grand Dixence Dam - Fionnay (Dixence) underground plant.

130. HIDROCENTRALA JABLONICA, Z. Stefanovic. Elektrotehniski Vestnik v 21 1953 p 76-81. Jablonica hydroelectric power plant situated underground. 6 generators driven by impulse turbines.

131. IL PROBLEMA GENERALE E LA MODALITÀ DI CONSTRUZIONE DELLE CENTRALI IN CAVERNA (THE GENERAL PROBLEM AND THE METHOD OF CONSTRUCTION OF THE POWER STATION AT CAVERNA), Carlo Semenza. Società Cooperativa Tipografica, Padova 1953. Discussion and illustrations.

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132. AUSTRALIA PUSHES SNOWY MOUNTAIN SCHEME. Engineering News-Record v 152 Jan 7 1954 p 46-47. News item - start of Guthega Pond project as part of Snowy Mountain program - underground power station on upper Tumut River one of first contracts let.

133. CIVIL ENGINEERING FEATURES OF KITIMAT PROJECT, J. S. Kendrick. Boston Society of Civil Engineers v 41 Jan 1954 p 88-112. General outline of project - brief statistics on underground power plant - diagrams showing method of excavation for main block of powerhouse.

134. IN SWEDEN, THE WORLD'S BIGGEST UNDERGROUND POWER PROJECT. Oil-Power (Socony Vacuum) v 29 Jan 1954 p 8-. Harspranget - 351' vertical penstock - tailrace tunnel 43' x 53' used to transport mechanical equipment.

135. Damador Valley Project Water Power v 6 Jan 1954 p 4-11 Feb p 61-66. Proposed Konar underground power plant, India - 7-1/2 mile tailrace tunnel.

136. CALANCASCA DEVELOPMENT. Water Power v 6 Feb 1954 p 44-52, 57. Sasselo Station in Switzerland - remote controlled.

137. HIGH-HEAD UNDERGROUND HYDRO PLANT. Power v 98 Feb 1954 p 85-89, 196. Kemano plant - prime factors in favor of underground construction - saving in penstock steel - security - foundation difficulties minimized - increased protection against snow and rock slides, forest fires - reduction of maintenance costs - continuous year around construction schedule - uniform year around working conditions - low cost of rock excavation due to recent advances - reduced cost of concrete.

138. LES TRAVAUX DE PERFORMATION, F. Auroy, et al. Travaux v 38 Mar 1954 p 143-55. Work carried out in connection with construction of Montpezat underground hydroelectric plant.

139. THE MONTPEZAT POWER PLANT. Water Power v 6 Mar 1954 p 117. Montpezat power scheme in France - Scheme employs underground plant.

140. TÅSAN POWER STATION, E. Eskilsson. ASEA Journal v 27 Mar 1954 p 39-43. Swedish Power Station Tåsan - 740 feet below ground level - transport tunnel 140 sq. ft., 2660 feet long - also serves as surge tunnel and air duct - plant fed by 4.2 miles of tunnel and has 2.8 mile tailrace tunnel, both 183 sq. ft. in area - penstock from inlet tunnel slopes 1 : 1, is concrete upper 2/3 and sheet metal pressure tube grouted in concrete lower 1/3 - control room, transformers and switchgear underground - electrical equipment.

141. KITIMAT. Water Power v 6 Mar 1954 p 89-99, Apr p 124-35. Discusses in detail the excavation of Kemano tunnel, shafts and power house - Part I tunnel and shaft - Part II power house. Reasons for going underground - to avoid long surface penstock - further steel saving by sharing penstock stresses with rock - avoid dangers to surface penstock from bombing, forest fires and snow or rock slides - poor surface foundation for piers and anchors - extend construction season - well illustrated.

142. KEMANO - KITIMAT HYDRO-ELECTRIC SCHEME. Engineering v 177 Apr 2 1954 p 430-32. Entire development with three paragraphs to operation of underground turbines.

143. BOLTED SUPPORT FOR TUNNELS. L. v. Rabcewicz Water Power v 6 April 1954 p 150-55, May p 171-75. Technique of roof bolting - its advantages, particularly when driving through unstable rock - theoretical principles of bolting - uses the Forcacava tunnel and underground power station in Brazil to illustrate practical applications.

144. SOME RECENT SWISS HYDRO-ELECTRIC SCHEMES. The Engineer v 197 Part I Apr 9 1954 p 518-20, Part II Apr 16 p 554-5, Part IV May 14 p 698-701, Part IX Jun 18 p 878-82, Part X Jun 25 p 914-18. Review of seven major hydroelectric projects in ten articles - Part I reviews Grande Dixence and Mauvoisin in general - two underground plants, Fionnay (Mauvoisin) and Fionnay (Dixence) - Part II, Fionnay (Dixence) - Part IV, Fionnay (Mauvoisin) - Part IX outlines Oberhasli development on Aare River - Innertkirchen, Handeck II, Grimsel underground plants described - possible future Grimsel II - Part X outlines scheme on Maggia River - Peccia, Cavergno, Verbano underground plants - Peccia and Cavergno chambers similar with 92 ft. width - horseshoe type section with spring line of arch at base of excavation - plain concrete arch - illustrations.

145. SOME ECONOMIC CONSIDERATIONS FOR UNDERGROUND POWER STATIONS, G. B. Schroyer, Minutes of Meeting, Edison Electric Institute, May 4 and 5 1954 app F. (See also Section B of this paper). Factors influencing selection of above or underground scheme - compares costs of 3 surface plants and 3 underground plants - lists unit costs - underground plants are Guayabo, El Salvador; Maithon, India; Ambuklao, Luzon, Philippines - all are low head "Swedish" type developments and have horizontal shaft Francis units.

146. POWER FROM BERSIMIS, J. P. Smallwood. Compressed Air Magazine v 59 May 1954 p 128-33. Bersimis River Project - 2 mi., 35 ft. lined tunnel.

147. TUMUT PONDS PROJECT OF SNOWY SCHEME. Commonwealth Engineer v 41 Jul 1954 p 473-76. General article on Eucumbene - Tumut portion of Snowy Scheme.

148. DEVELOPMENT OF AGNO RIVER WITH SPECIAL REFERENCE TO DESIGN OF AMBUKLAO PROJECT, J. O. Lahoz. Philippine Engineering Record v 15 Aug 1954 p 6-16. 425' rockfill dam and underground plant.

149. THE HARSPRANGET POWER STATION AND THE SWEDISH 380 KV SYSTEM. ASEA Aug 1954. 143 page special issue of ASEA discussing all phases of electrical equipment and machinery of Harspranget plant and 380 KV transmission system which it feeds - color photo showing gunited roof and crane supported on columns.

150. POWER SYSTEM OF DAMODAR VALLEY PROJECT, T. Saran, N. M. Jalan. Indian Journal Power & River Valley Development v 4 Aug 1954 p 25-27, 36. General system - Konar and Maithon plants.

151. SWEDISH UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, J. F. Hagrup. Institution of Civil Engineers Proceedings v 3 Aug 1954 p 321-44. Sweden's water power resources - design of underground water power stations, blasting tunnels and underground stations - costs for power stations with underground machine hall.

152. FORCACAVA HYDRO-ELECTRIC SCHEME, L. V. Rabcewicz and Portland P. Fox. Water Power v 6 Sep 1954 p 353-54. Letters to editor of Water Power discussing article by Prof. Rabcewicz which appeared in Water Power v 5 Sep, Oct, Nov, 1953 - discuss excavation and construction methods used - name of plant changed to "Nilo Pechanha."

153. HYDRO-ELECTRID DEVELOPMENT IN PORTUGAL, PART II. English Electric Journal, v 13 Sep 1954 p 303-13. River Cavado Power Scheme includes Salamonde and Canicada underground plants - latter under construction.

154. QUED AGRIOUN DEVELOPMENT, H. Castelnau and Y. Haller. Technique Moderne- Construction v 9 Sep 1954 p 307 (Abstracted in Water Power v 7 Feb 1955 p 67). Qued Agrioun Development, Algeria, includes Darquinah power station (sometimes called Ahrzerouftis).

155. SALAMONDE HYDRO-ELECTRIC STATION IN PORTUGAL, M. Ebersberger. The Brown Boveri Review v 41 Oct 1954 p 359-70. Description of power plant with emphasis on electrical equipment -

photos and diagrams - plant of Swedish type with power plant just below dam - separate valve chamber with bypass tunnel around power plant - access shaft to power chamber contains elevator, ventilation duct, stairs, control and signal cables, bus bars between generators and transformers and a space for lowering equipment - gallery from shaft leads to surge chamber and to tailrace gate chamber - transformers on surface - power chamber concrete lined - crane rail beams supported on cavern sides and tied to arch roof.

156. 230-KV PIPE-TYPE CABLE IS LINK IN SAO PAULO SUPPLY, S. T. Franks. Electrical World v 142 Nov 29 1954 p 76-79, 166. Electrical equipment for Cubatao underground plant.

157. ALA DEVELOPMENT, ITALY. L'Energia Elettrica v 30 Nov 1954 p 763 (Abstracted in Water Power v 6 Aug 1954 p 318-19). Ala hydroelectric plant near Trento, Italy, including underground power station - two tailrace tunnels 130 m long discharge into open canal outdoor switchyard and transformer station.

158. THE NECHAKO - KEMANO - KITIMAT DEVELOPMENT (KEMANO UNDERGROUND), F. T. Mattias. Engineering Journal v 37 Nov 1954 p 1398-1412. Part of symposium on complete project by various authors - detailed review of construction details, plant, methods, equipment, etc. used in excavation and concreting of Kemano power chamber, valve chamber, access tunnels, pressure conduits, etc.

159. THE SWITCHGEAR OF VINODOL UNDERGROUND HYDRO-ELECTRIC POWER STATION IN YUGOSLAVIA, M. Ebersberger. The Brown Boveri Review v 41 Nov 1954 p 405-17. Problems and advantages of underground power plants, particularly with respect to electrical equipment - switchgear and transformers underground - 110 KV cable in inclined access tunnel - list of underground plants.

160. DÖNJE POWER STATION, O. Sjöberg ASEA Journal v 27 Nov-Dec 1954 p 159-76. Electrical equipment including generators, transformers, switchgear, control equipment, fire extinguishing equipment and air conditioning plant - erection hall, control building and switchgear on surface - machine hall has false arched ceiling and false walls - cranes supported on columns.

161. THE SALAMONDE HYDRO-ELECTRIC SCHEME, L. H. Gomes Fernandes. Water Power v 6 Nov 1954 p 408-18, Dec p 449-56. Part one, entire Salamonde scheme - Part two, underground power plant - well illustrated.

162. BUILDING UNDERGROUND PAYS IN SWEDEN, G. Westerberg, Engineering News-Record v 153 Dec 9 1954 p 33-39. Reasons why under ground power plants, industrial plants, etc. are so economically constructed in Sweden - development of techniques, and mass production basis of tunneling.

163. LA CENTRALE SOUTERRAINES DE RANDENS, Olivier-Martin. Annales de L'Institut Technique du Batiment et des Travau Publics Dec 1954 p 1263-73. Factors in choice between underground and surface scheme - detailed description of Isere Arc station.

164. OVER 7 MILLION YARDS OF ROCK EXCAVATED FOR TWO POWER PROJECTS, Tore Nilsson Engineering News-Record v 153 Dec 16 1954 p 41-43. Excavation of Kilforsen and proposed Stornorrfors plants - rock walls at Kilforsen exposed with roof secured by rock bolts and gunited.

165. CONSTRUCTION UNLIMITED, BOOK ONE: DAMS AND POWERHOUSES. Morrison-Knudsen Company, Inc. 1954-1955. Brief and general description of 3 underground plants constructed in whole or in part by M-K Kemano, Nilo Pecanha, Cubatao - illustrated.

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166. OBERHASLI POWER SCHEMES, Engineering v 179 Jan 7 1955 p 13-16, Jan 14 p 50-53. Features of Swiss developments including Innertkirchen and Handeck II underground power stations - plans and profiles of schemes - powerhouse pictures and sections - Innertkirchen first Swiss underground station 1942 - features of site and good rock made underground economic chamber unsupported but fully lined - false ceiling for seepage - details turbines - shaft in 3 sections to reduce station height - tailrace tunnel submerged - penstock design discussed - Handeck II underground because no suitable outdoor site available and rock is hard and compact - separate valve chambers in case of valve housing rupture - machine hall unsupported but fully lined, 20" arch thickness - shaft in 3 sections to minimize station height.

167. LES CLÉES POWER STATION. Bulletin de L'Association Suisse des Electriciens v 46 Feb 1955 p 149. Swiss hydro-electric plant, part of Orbe River Development.

168. SWEDISH POWER PLANT FORCED "BOLD TECHNICAL SURGERY," Holger Lundbergh. The Plant Feb 1955 p 55-57. Kilforsen power station with emphasis on size and importance - manufacturers of major equipment - supply tunnel used as log chute.

169. BERSIMIS: ANOTHER KEMANO. Engineering News-Record v 154 Mar 3 1955 p 28-29. Broad description of hydro-electric project in Quebec - two dams, 7-1/2 mile tunnel, large underground power plant.

170. L'AMÉNAGEMENT DE LA CHUTE DE MONTPEZAT. Génie Civil v 75 Mar 15 1955 p 101-4. Utilization of head of Montpezat, Loire River - Installation of underground plant.

171. CUBATAO - BRAZIL'S NEW 260 MW UNDERGROUND HYDRO STATION, Erik A. Johanson and A. G. Clarkson. Electric Light and Power v 33 Mar 25 1955 p 94-98. Excavations are entrance tunnel, valve chamber, machine hall, air shaft, cable tunnel and tailrace tunnel - concrete arch and walls - transformers in machine hall - ventilating air through access tunnel and out exhaust shaft - tailrace cooling water.

172. SWISS UTILITIES BUILD HUGE "WINTER POWER" HYDRO PROJECT, Alexandre Verrey. Electric Light and Power v 33 Mar 25 1955 p 110-17. Swiss and French Companies develop Dranse de Bagnes in Alps as joint venture - dam and two powerhouses will develop 330,000 kw - Fionnay (Mauvoisin) underground power station has world's highest head Francis turbines - data is general - illustrations.

173. PRESENT TRENDS IN THE DESIGN OF PRESSURE TUNNELS AND

SHAFTS FOR UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, C. Jaeger. Proceedings of the Institution of Civil Engineers Part I v 4 Mar 1955 p 116-200 (See Section B of this paper.)

174. THE SOG DEVELOPMENT, Steingrimur Jonsson, Water Power v 7 Mar 1955 p 84-92. Irofoss plant of Iceland - tailrace surge chamber - lined tailrace flow tunnel - machine hall, fully lined and false roof - vertical shaft at one end leads to outdoor switch gear and control room - contains elevator, stairs, conduits, ventilation - also auxiliary access.

175. PUMPED STORAGE - SWISS PROJECT FOR HYDRO POWER NEARS COMPLETION. Engineering News-Record v 154 Apr 21 1955 p 75-76. News item Oberaar - Grimsel project nears completion with final phases of construction of Oberaar underground power plant.

176. HYDRO-ELECTRIC DEVELOPMENT OF THE QUED AGRIOUN, J. Lordet, H. Ouquennois, J. Guilhaman. Travaux (Edition of Science et Industrie), special supplement, Fifth International Congress on Large Dams. May 1955 p 203-206. Darguinah power station on the Qued Ahrzerousftis in Algeria - photos and drawings - reason for underground - main river has neither penstock site (very flat) nor power house foundation - power house 200' underground in rocky tributary canyon with tailrace tunnel to develop full head - poor ground - heavily supported - fully concrete lined - valves in machine hall - transformers above ground - penstock on surface except for lower 200' in 45^{o} shaft.

177. POWER FROM GLENS. Civil and Structural Engineers Review v 9 May 1955 p 218-24. Glen Moriston plant, Scotland - 20,000 KW.

178. GOSCHENALP DEVELOPMENT, W. Eggenberger. Bulletin de l'Association Suisse des Electriciens v 46 Jun 8 1955 p 725 (Abstracted in Water Power v 8 Jan 1956 p 39) 7' dia. by 3500' long steel lined pressure shaft - transformers underground - proposed project.

179. AUSBAU DES SPEICHERWERKES JABLANICA, Dragan Caric Der Bauingenieur v 30 Jun 1955 p 201-207 - cavern fully concrete horseshoe lined - separate drained valve chamber - transformers underground.

180. THE NEW TECHNIQUE OF UNDERGROUND HYDRO-ELECTRIC POWER STATIONS, C. Jaeger. The English Electric Journal v 14 Jun 1955 p 3-29 (See Section B of this paper.)

181. DEVELOPMENTS IN THE ÅNGERMAN CATCHMENT. Water Power v 7 Jun 1955 p 202-13, Jul p 247-53, Aug p 292-300. Part One, Ångerman River catchment in central Sweden - Kilforsen power station - 12,000' tunnel, 1240 sq. ft. area, unlined - penstocks vertical steel lined shafts, 17.7' dia. - sides of chamber exposed - cranes on concrete columns - transformers and switchgear in separate hall - cables to surface by 3' dia. shafts over each transformer - tailrace tunnel 8800' long and 1900 sq. ft. area - machine hall roof bolted, wire netted and gunited - Part Two, mechanical electrical equipment of Kilforsen station - Part Three, Lasele underground plant.

182. SWEDES MAKE ROCK TUNNEL HISTORY, Waldo G. Bowman, Engineering News-Record v 155 Sep 1 1955 p 34-37, 40-44. Latest projects in a continuing underground program - two hydro plants and an air raid

shelter make new advances in sizes of tunnels and speed of driving - hydro plants are Harresele and Stornorrfors on the Ume River - Harresele plant - discharge tunnel 49 x 59 x 11,154 ft. long - transformers outdoor - vertical 23 ft. dia. steel penstocks embedded in concrete - Stornorrfors - discharge tunnel 53 x 77 ft., 2-1/2 miles long.

183. EXCAVATION OF NO. 4 POWER STATION, KIEWA, R. I. Rankin, Chemical Engineering and Mining Review v 47 Sep 10 1955 p 489-93. Design and construction considerations for underground plants - geology of site excavation methods - poor rock resulted in concreting arch and walls at Kiewa - some construction schedule problems peculiar to underground plants.

184. HYDRO-QUEBEC IS DEVELOPING MORE THAN HORSEPOWER AT BERSIMIS, Arthur J. Fox, Jr. Engineering News-Record v 155 Nov 17 1955 p 34-39. Bersimis Project in eastern Quebec - Includes two dams, 7.5 mi. tunnel, eight penstocks inclined at 50 degrees, to horizontal underground powerhouse, 47 x 75 ft. tailrace tunnel, eight 150,000 hp units - cooling water from penstocks - concrete lined arch roof, suspended ceiling, bare walls - 6 x 8 ft. cable tunnel to outdoor transformers and switchyard.

185. KEMANO PENSTOCK TUNNEL LINER BACKFILLED WITH PREPACKED CONCRETE, R. E. Davis, G. D. Johnson, G. E. Wendell. Journal of the American Concrete Institute v 52 Nov 1955 p 287-308. Concrete control and placing methods used in backfilling shaft - design assumptions and results.

186. KINGS RIVER WILL BE BIG ADDITION TO CALIFORNIA HYDRO. Electrical West v 115 p 62-64 Dec 1955. P.G. & E.'s proposed hydroelectric development on Kings River - general information and statistics concerning entire development - profile, plans and sections of Haas underground power house - special topographic features and excellent rock make underground scheme economic - vertical multiple jet impulse turbines.

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187. HYDRO-ELECTRIC WORK IN SCOTLAND. Engineering v 181 Jan 13 1956 p 40. Summary of recent hydro-electric work - Clachen and Mullardoch power plants commissioned.

188. HIGH-SPEED TUNNELING TECHNIQUES. Water Power Jan 1956 p 8-16. Breadalbane scheme, Scotland - St. Fillans powerhouse - underground for scenic reasons - 10' dia. vertical shaft concrete lined to horizontal steel lined tunnel - tunneling methods.

189. THE UPPER TUMUT WORKS, D. E. Campbell, A.N.G. Bray, I. L. Pinkerton, A. C. H. Frost. Institution of Engineers, Australia (Journal) v 28 Jan-Feb 1956 p 1-27. A symposium - half devoted to T-1 underground plant - design criteria for access and plant dimensions - ventilation and lighting.

190. THE UPPER QUED DJEN DJEN PROJECT, F. Cordelle and M Durand. Water Power v 8 Feb 1956 p 46-54. Algerian development which includes one dam and two power-plants - Mansouria underground - discharges into Mediterranean - valve chamber, machine hall and substation are separate chambers.

191. VINSTRA. Water Power v 8 Mar 1956 p 86-93, Apr 150-56. Underground plant of Vinstra development in Norway - three separate tunnels for transportation, cables, and tailwater - penstock tunnel isolated from station by self-closing steel bulkhead - connecting tunnel bypasses machine chamber to tailrace - transformer hall parallel to machine hall.

192. NORTH OF SCOTLAND HYDRO-ELECTRIC SCHEMES - THE GLEN SHIRA SCHEME. The Engineer v 201 Apr 20 1956 p 364. Glen Shira scheme includes an underground pressure shaft and Clachan power station - Clachan is a cut and cover plant with concrete lined walls and arch roof - Review of pressure shaft design.

193. BERSIMIS - LAC CASSE, F. Rousseau. The Engineering Journal v 39 Apr 1956 p 373. Cavern has concrete arch roof - transformers outside - Discusses steel lined penstocks and powerhouse excavation - photos - valves in machine hall - underground for economic reasons.

194. STEEL LININGS FOR PRESSURE SHAFTS IN SOLID ROCK, E. W. Vaughn. Journal of the Power Division, Proceedings of the American Society of Civil Engineers v 82 Apr 1956. Factors in design of steel linings for pressure shafts for underground hydroelectric plants - methods used in design of two such shafts in Brazil, Nilo Pecanha and Cubatao discussed in detail and general features compared with those of other shafts constructed elsewhere - general approach to problem together with pertinent details of design and construction.

195. USINES HYDRO-ELECTRIQUES SOUTERRAINES INSTALLATION ET TRANSFORMOTEURS ENTRE LES GROUPES, B. Level. Electricite. Ed. Science et Industrie May 1956 p 119-21.

196. THE MONTPEZAT SCHEME, Walter T. Storey. Water Power v 8 May 1956 p 167-73, Jun 226-30. Montpezat plant in France - Part I generally describes power station, Part II dams, reservoirs, tunnels - turbine valves in separate gallery - switching above ground - access gantry in inclined tunnel above busbars.

197. HANABANILLA: CUBA'S HYDRO PROJECT. Engineering News-Record v 156 Jun 14 1956 p 73-76. Cuba's first hydro project of magnitude - underground scheme saved estimated $3,000,000 compared to surface plant - inclined access tunnel - transformers and switchyard located above ground - 4-1/2 mile tailrace tunnel.

198. THE NEW STATIONS OF THE KRANGEDE POWER COMPANY. Water Power v 8 Jul 1956 p 264-68, Aug 306-11. Work of Krangede Power Company in Angerman catchment in Sweden - machinery of Storfinnforsen plant and brief general outline of Ramsele plant under construction.

199. KARIBA HYDRO-ELECTRIC SCHEME ON THE ZAMBESI RIVER. The Engineer v 202 Aug 3 1956 p 154-6. Kariba Dam and underground power station with future second underground plant on Zambezi River, Rhodesia - Power plants to be under right and left abutments of dam - principal elements of powerplants and dam - main contracts.

200. DEVELOPMENT OF WATER-POWER RESOURCES IN THE NORTH OF FINLAND, V. Verjola. Water Power. v 8 Aug 1956 p 295-302 - Jumisko

first underground plant in Finland - exposed rock walls - remotely controlled from plant 110 miles away - Swedish type development.

201. UNDERGROUND PLANT UPS MEXICO'S KW. Electrical World v 146 Sep 24 1956 p 48-49. Tingambato hydro plant on Miguel Aleman System - walls reinforced concrete - vertical welded penstock.

202. THE AURA DEVELOPMENT. Water Power v 8 Sep 1956 p 327-34, Oct 389-96. Aura power station, Norway's largest - Part I describes development in general, Part II power plant - 2 machine halls - tunnel branches at surge chamber - machine halls 165 ft. apart - false ceilings - south hall has separate valve chamber.

203. HYDROELECTRIC DEVELOPMENT ON THE NORTH FORK OF THE KINGS RIVER, W. R. Johnson, D. P. Dinapoli, J. B. Cooke. Electrical Engineering v 75 Oct 1956 p 900-905. Pacific Gas and Electric Company development in California - four dams, three power plants - Haas underground - largest vertical shaft impulse turbines in U.S. - two of highest head plants in U.S. - Haas has rock bolted and gunited arch roof - tailrace used as access - turbine valve in main chamber.

204. PERU HARNESSES A PLUNGING RIVER. Engineering News-Record v 157 Nov 1 1956 p 51-54. The Canan del Pato Project - general outline - four 1430 ft. deep 10 ft. dia. shafts sunk at powerhouse site for construction purposes - three will be used for power - double overhung Pelton turbines.

205. TUNNEL BLASTING AT HARRSELE, Ragnar Ahlström. Water Power v 8 Nov 1956 p 407-12. Tunnel driving techniques for 58 x 32 ft. tailrace tunnel.

206. CUBA'S UNDERGROUND POWERHOUSE AND DAMS ATTRACT INTERNATIONAL BIDDING. Engineering News-Record v 157 Dec 6 1956 p 274-75. Unit prices and comparative total bids for powerplant and dams.

207. ASPECTS OF RECENT DEVELOPMENT OF HYDRO-ELECTRIC POWER IN SWEDEN, Gunnar Isaksson. Fifth World Power Conference 1956, Section H/10, Paper 54 p 1-11. Nearly all plants "Swedish" scheme with powerhouse near intake and long tailrace tunnel - penstocks vertical - minimum of 2 independent exits - trend from shaft access to 10% grade transport tunnel.

208. HYDRO-ELECTRIC DEVELOPMENT IN INDIA, M. Hayath, K. L. Vij. Fifth World Power Conference 1956 Section H Paper 179 H/29 p 1-13. Progress and plans - Damodar Valley Project includes Maithon and Konar underground - projected Koyna Project calls for three-stage development underground.

209. KEMANO - ADVANCES IN DESIGN AND CONSTRUCTION, F. L. Lawton. Fifth World Power Conference 1956 Section H/25 Paper 154 p 1-14. General review of Kenney Dam, power tunnel, pressure conduits, powerhouse, generating units, transmission lines.

210. PROGRESS ON THE SNOWY MOUNTAINS SCHEME, The Snowy Mountains Hydroelectric Authority. Fifth World Power Conference 1956 Section H/43, Paper 259 p 1-16. Background, description, utilisation of

(Contd. on p. 1350-36.)

SECTION C

LIST OF WORLD'S UNDERGROUND HYDROELECTRIC PLANTS WITH GENERAL DATA AND REFERENCES

This list of plants and the tabulated data have been obtained from the references and, in many cases, from foreign engineers. The writers thank the foreign engineers for their cooperation.

Explanation of column headings:

(1) Plants are listed alphabetically by country and by plant name.

(2) Number of turbines and type. The type of turbine unit is abbreviated with following legend: F = Francis, K = Kaplan, P = Pelton, d = double overhung, h = horizontal shaft and v = vertical shaft.

(3) Plant capacity in megawatts.

(4) Maximum static head in feet.

(5) Dimensions of powerhouse excavation in feet, Length x Width x Height. Machine hall height, above generator floor to crown of arch, is shown in parentheses.

(6) Number of reference in the Bibliography, Section D.

(7) Year of initial operation. Where year is later than 1957, plant is under construction or proposed.

Country and Plant (1)	Turbines No. and Type (2)	Plant Capacity MW (3)	Static Head Ft. (4)	Powerhouse Chamber L x W x H (5)	Reference Number (Section D) (6)	Year Initial Oper. (7)
ALGERIA					116	
Darquinah (Agrioun)	2 Pv	66	1236	239 x 106 x 85	154, 173, 176, 180	1951
(Ahrzerouftis)	1 Fv	5	555			
Mansouria	2 Ph	100	1958	219 x 46 x 59	190	196-
AUSTRALIA					25, 27, 32, 34, 96	
Barren Falls	3 Fh	3.8	415	81 x 25 x (22)		1935
Kiewa No. 4	4 Fv	64	653	130 x 40 x 72	28, 29, 30, 183	1955
Kiewa No. 1	6 Pv	96	1730	244 x 45 x 78	28, 29, 30	1961
M-6 (Snowy Mt.)		1000±			112, 210	196-
M-7 (Snowy Mt.)					112, 210	196-
T-1 (Snowy Mt.)	4 Fv	320	1100	305 x 59 x 105	32, 101, 112, 132, 147, 173, 180, 189, 210	1958
T-2 (Snowy Mt.)	4	280	860		112, 210	1962
Tully Falls No. 2	4 Ph	72	1485	227 x 44 x (44)	183	1957

BRAZIL						
Cubatao	6 Pv	465	2365	397 x 65 x 132	122, 156, 165, 171, 180, 194	196-
Juquia Guassu	2	25			159	
Nilo Pecanha (Forcacava)	6 Pv	330	1130	377 x 62 x 85	68, 104, 117, 123, 143, 152, 165, 180, 194	1954
Paulo Afonso	3 Fv	180	273	203 x 56 x 111		1952
	6 Fv	360	273	360 x 59 x 118		Ult.
CANADA						
Bersimis	8 Fv	895	875	565 x 65 x 80	146, 169, 173, 180, 184, 193	1956
Chute-des-Passes	5 Fv	746	650	460 x 70 x 110		1959
Kitimat	8 Pv	600	2500	700 x 82 x 139	69, 81, 82, 86, 87, 91, 94, 97, 100, 102, 108, 109, 111, 113, 115, 125, 127, 128, 133, 137, 141, 142, 158, 165, 173, 180, 185, 209	1955
	16 Pv	1200		1140 x 82 x 139		Ult.
CHINA (FORMOSA)						
Lung-Chien	1 Ph	48.6	2933	150 x 47 x 58		1958
Tung-men	3 Fv	21.8	512	163 x 36 x 73		1955
CUBA						
Habanilla	3 Fv	45	870	120 x 40 x 75	197, 206	1959
EGYPT						
High Aswan	16 Fv	2880	400	2 chambers, ea. 700 x 70 x 100(75)		196-
EL SALVADOR						
Guayabo	3 Fh	45	184	169 x 37 x 62	43, 62, 145, 159	1954
FINLAND						
Jumisko	1 Fv	30	295		200	1955

Country and Plant (1)	Turbine No. and Type (2)	Plant Capacity MW (3)	Static Head Ft. (4)	Powerhouse Chamber L x W x H (5)	Reference Number (Section D) (6)	Year Initial Oper. (7)
FRANCE					96	
Bathie	6 Pv	496	3963	405 x 81 x 106		1960
Baton	2 Ph	5.2	3428	85 x 20 x 30	51	1925
	1 Ph		1843			
Bois	2 Ph	48.5	1179	261 x 62 x 67		196-
Brommat	6 Fv	186	853	262 x 72 x 103	9, 51, 57, 121, 173, 180	1932
Carla	2 Fv	30	466	243 x 43 x 77		1958
Froges	8 Ph		4822	738 x 66 x 70		
Maison Blanche	1 Pv	7.7	984	69 x 29 x (37)		1946
Montahut	2 Ph	64	2017	175 x 48 x 49		196-
Montpezat	2 Ph	117	2083	197 x 44 x 88	138, 139, 173, 180, 196	1954
Peyrat-le-Chateau	3 Fv	63	830	138 x 36 x 57	64	1952
Rabodanges	1 Fv	6.6	164	37 x 33 x 36		1959
Randens (Isere-Arc)	4 Fv	136.2	498	357 x 55 x 103	116, 163, 170, 173, 180	1954
St. Cassien	2 Fv	24	382	111 x 38 x 57		196-
St. Vincent Durfort	4 Fv	240	864	384 x 61 x 79		196-
Serre-Poncon	4 Fv	320	410	328 x 55 x 88		1960
Tolla	2 Fh	10.6	682	113 x 36 x 51		196-
Treignac	2 Fv	33	540	98 x 33 x 56		1952
Valabres	2 Fv	30	664	108 x 36 x 56		1957
Vaujany	1 Ph	42.5	2431	102 x 51 x 59		196-
GERMANY					22, 23, 96	
Brechbergmuehle					51, 173, 180, 203	1907
Friedigen		2.6	177		51	
Partenstein						
GUINEA (FRENCH)						
Amaria	6 Fv	450	270	390 x 56 x	176	196-
Souapiti	4 Fv	300	400	230 x 41 x 92	176	196-
ICELAND						
Irofoss	2 Fv	31	124	137 x 40 x	70, 174	1953
	3 Fv	46.5				Ult.

					103	
INDIA						
Konar	2 Fv	40	600		119, 135, 150, 208	196-
Koyna	4	240	1680	545 x 85 x	121, 208	196-
	4	280				Ult.
Maithon	3 Fh	60	138	235 x 45 x 70	135, 145, 150, 208	1957
ITALY						
Agordo	2 Fv	26	492	114 x 50 x 92	35, 41, 96	1940
Ampezzo (Lumiei)	3 Ph	59	1565	174 x 51 x 79	48, 131, 173, 180	1947
Ala	2 K	50	89	140 x 44 x 61	75, 157	1952
Avise	3 Ph	150	3410	242 x 61 x (56)		1954
Barcis ($\frac{1}{2}$ under)	1 Kv	12	190	97 x 47 x 82		1954
Belviso	2 Pdh	63	1740	304 x 54 x		1957
Castelbello	3 F	85	968		173, 180	1950
Cencenighe	2 Fv	30	662	77 x 38 x 95	51, 131	1939
Coghinas					(1st in Italy) 51, 131	1927
Cotilia	4	67	116	(half under)	67	1942
Flumendosa					7, 51, 131	1928
Gardona	1 Fh	14	492	115 x 48 x 67		1955
Glorenza	4 Ph	95	1952			1949
Hone	1 Kv	19	121	74 x 42 x (62)		1947
Lana da Adige	3 Ph	120	1575	220 x 56 x (59)		1953
Lovero	2 Fv	44	357	167 x 62 x 83		1948
Malga Ciapela	1 Ph	19.4	1860	109 x 47 x 66		1956
	(3 Pumps)	2.4				
Mera	3 Fv	73	1080		63, 173, 180	
Monte Argento	3 Fv	66	164	241 x 34 x (49)	51, 67, 173, 180	1950
Montorio	3 Fv	115	860	197 x 55 x (62)	51, 67, 173, 180	1955
Narni	2 Kv	44	92	184 x 59 x (52)		1957
Ponte Gardena	3 Fv	60	200			1938
Pontey	2 Fv	34	228	149 x 56 x (59)		1950
Premadio	2 Ph	150	2204	264 x 98 x 85		1955
Provvidenza	2 Fh	100	876	393 x 52 x (59)	51, 173, 180	1949
	Pump	50				
Quart	2 Fh	50	257	168 x 63 x 57		1958
San Giacomo	3 Pdh	210	2175	310 x 53 x (48)	51, 173, 180	1948
Santa Giustina	3 Fv	108	600	200 x 93 x (20)	63, 84, 173, 180	1951
Santa Massenza	6 Pdh	355	1750	650 x 95 x 92(66)	63, 121, 173, 180	1953
No. 1 and 2	1 Fv	27	650			1957
Somplago	5 Fv	275	950	292 x 82 x 114		1957
Sondrio	2 Pv	140	2177	144 x 85 x (56)		1959
Sonico-Cedegolo	3 Fv	74	797		63, 173, 180	1952

Country and Plant (1)	Turbine No. and Type (2)	Plant Capacity MW (3)	Static Head Ft. (4)	Powerhouse Chamber L x W x H (5)	Reference Number (Section D) (6)	Year Initial Oper. (7)
FRANCE (CONTD)						
Soverzene	4 Fv	220	930	243 x 83 x 131	78, 173, 180	1950 1956
Stanga	2 Fv	30	490			1943
Stazzona	2 Fv	35	305	107 x 56 x 87		1938
Zappello	2 Fh	10	1250	154 x 31 x 45(30)		1931
JAPAN						
Iyogawa	2 Fv	6	151	53 x 33 x 74		1947
Kurobe No. 4	3 Pv	258	1935	310 x 85 x 135		1960
Miboro	2 Fv	215	652	210 x 83 x 149		1961
Okutadami	3 Fv	360	635	289 x 76 x 131		1960
Sudagai	2 Fv	46	274	115 x 56 x 96		1955
MEXICO						
El Cobano	2 Fv	52	1000	80 x 41 x 85(43)	159	1955
El Encanto	2 Fh	10	260	118 x 45 x 45(36)	159, 201	1951
El Zotolin	2 Fh	0.3	232	40 x 16 x 35(20)		1934
Tingambato	3 Fv	150	1127	131 x 55 x 114		1956
NORWAY					54, 96	
Aumfossen	1 Fv	26	82			1959
Aura	4 Ph	120	2560	405 x 56 x 59	202	1953
	3 Ph	168	2560	310 x 56 x 49	202	1956
Balmi	2 Fv	36	510			1958
Bagn	2 Fv	44	292	105 x 43 x 33		1960
Bardufoss	2 Fv	30	174	154 x 43 x 43		1953
Bjorkasen	3 Fh	5	272	85 x 36 x 36		1921
Brattingfoss	1 Fh	10	475	128 x 33 x 33		1955
Dynjanfoss	3 Fv	33	262	131 x 33 x 33		1951
Fiskumfoss	3 Fv	39	115			1946
Fjaeremsfoss	3 Fv	26	89	170 x 36 x 39		1957
Fortun	6 Ph	108	3280			1960
Fosse	1 Fv	22	328	89 x 43 x 39		1954
Gronsdal	2 Fv	24	590	115 x 59 x 56		1949
Hemsil I	2 Fv	59	1770			1959

Hemsil II	2 Fv	76	1210			1959
Hjartdola	2 Pv	104	1930			1958
Hol II	1 Fv	28	158		212	1957
Hol III	2 Fv	60	318			1957
Hakvik	2 Fv	9	720	79 x 26 x 30		1954
Haverstad	2 Fv	70	270	112 x 40 x 40		1957
Innset	2 Fv	90	582			1960
Kaggefoss	4 Fv	64	230			1950
Kurasfoss	2 Fv	9	150			1952
Lyse I	3 Ph	90	2080	310 x 40 x 40	173, 180, 212	1951
Lyse II	2 Ph	80	2080			1959
Mar	5 Ph	180	2700	550 x 56 x 59	73, 173, 180	1954
Matre	3 Ph	96	1710	375 x 65 x 53		1955
	2 Fv	86	1530			
Nea	3 Fv	140	1230			1960
Niingen	1 Ph	12	1650			1953
Porsa	1 Fv	12	705			1959
Rjukan III	1 Fv	25	156			1955
Rjukan IV	1 Fv	35	161			1957
Rosaga N.	6 Fv	120	800	510 x 46 x 40		1955
Rosaga Ov.	3 Fv	150	430			1961
Steinsfossen	2 Fv	46	190			1957
Solbjornsvann	2 Fh	7	295	138 x 43 x 40		1956
Svartelven	2 Fv	12	328			1960
Svelgen II	2 Ph	22	1620			1958
Svelgfoss III	2 Fv	90	226			1958
Saheim	1 Fh	6	590			1916
Tafjord II	3 Ph	42	1310	220 x 30 x 36	173, 180, 212	1952
Tafjord III		16			212	1958
Tessa III	1 Fv	8	558			1956
Tinfos I	2 Kh	38	98	122 x 39 x 40		1955
Tistedalen	2 Fv	21	216			1955
Hanefossen	1 Fv	10				1960
Tokke I	4 Fv	400	1280			1961
Tussa	2 Ph		2080			1960
Tyin	5 Ph	168	3500	377 x 52 x 63	51, 73	1945
Vinstra N.	4 Fv	200	1450	262 x 40 x 82	73, 173, 180, 191	1955
Vinstra Ov.	2 Fv	140	1080			1959
Oksenelvane	2 Ph	26	1280	220 x 40 x 36		1954
Abjora	3 Ph	91	1410	328 x 56 x 44		1951

Country and Plant (1)	Turbine No. and Type (2)	Plant Capacity MW (3)	Static Head Ft. (4)	Powerhouse Chamber L x W x H (5)	Reference Number (Section D) (6)	Year Initial Oper. (7)
PERU						
Canon del Pato	6 Ph	150	1350	367 x 36 x (49)	173, 180, 204	1957
PHILIPPINES						
Ambuklao	3 Fh	75	572	230 x 42 x 66	107, 110, 145, 148	1956
PORTUGAL						
Alto Rabagao		60	618			1964
Canicada	2 Fv	60	397	129 x 60 x 101(50)	153	1955
Miranda	3 Fv	150	216	262 x 69 x 113(57)		1960
Picote	3 Fv	180	251	289 x 55 x 115(48)		1957
Salamonde	2 Fv	42	416	120 x 38 x 96(45)	153, 155, 159, 161	1953
RHODESIA						
Kariba (right)	6 Fv	600	300	460 x 75 x 125	199	1960
Kariba (left)		600	300	460 x 75 x 125	199	Future
SCOTLAND					25, 120	
Ceannacroc	2 Fv	20	299	96 x 47 x 82(46)	85, 173, 180	1956
Clachen (cut and cover)	1 Fv	40	960	80 x 56 x 75(53)	45, 173, 180, 187, 192 213	1955
Glen Moriston	2 Fv	32	311	104 x 46 x 76(41)	85, 173, 177, 180	1957
Mullardoch	1 Fv	2.4	90	36 x 26 x 45(28)	19, 79, 88, 95, 187	1955
St. Fillans	1 Fv	21	830	50 x 34 x 68	188	1957
SPAIN					96	
Tranco de Beas	2 Fv	41	360	131 x 46 x 85	38	1947
Barasona	2	22			159	1948
SWEDEN					21, 25, 52, 53, 55, 56, 96, 98, 151, 207, 211	
Alfta	2 Fv	20	201	67 x 31 x 69(64)		1945
Blasjon	1 Fv	57	294	83 x 41 x 90(54)		1957
Brannalven	2 Fv	50	345	-		1962
Balforsen	3 Kv	105	102	253 x 53 x 102(53)		1958
Donje	2 Kv	41	107	125 x 39 x 82(39)	160	1953

Edensforsen	2 Kv	60	92	157 x 46 x 85(43)		1956
Forsmo	4 Kv	140	112	302 x 53 x 108(56)	181	1948
Glava	1 Fv	16	320	53 x 34 x 62(37)		1958
Grundfors	2 Kv	88	110	195 x 49 x 102(49)		1958
Gulsele	3 Kv	65	97	167 x 44 x 82(43)		1955
Harrsele	3 Fv	200	179	257 x 52 x 106(47)	182, 205	1957
Harspranget	3 Fv	350	350	342 x 61 x 102(52)	42, 51, 58, 59, 74, 76, 80, 92, 105, 124, 134, 149, 173, 180	1951
Hjalta	3 Fv	175	282	197 x 43 x 104(48)	66, 173, 180	1949
Hojum	2 Kv	94	105	217 x 59 x 108(62)	89, 173	1942
Holjes	2 Fv	110	289			1962
Junsterforsen	1 Kv	26	36	14 x 26 x 25(12)		1961
Jarpstrommen	3 Fv	120	217	187 x 36 x 80(49)	20	1944
Jossefors	1 Kv	25	82	49 x 38 x 116(91)		1951
Kilforsen	3 Fv	240	325	318 x 59 x 108(59)	51, 74, 164, 168, 181	1953
Korsselbranna	1 Fv	70	351	39 x 85 x 98(53)		1961
Krokstrommen	3 Fv	80	195	184 x 43 x 72(36)		1952
Kvistforsen	2 Kv	130	164	253 x 59 x 98(49)		1960
Krangede	6 Fv	227	197	322 x 41 x 92(56)	11, 20, 173, 180	1936
Laforsen	2 Fv	32	114	131 x 43 x 117(79)	74, 181	1953
Lasele	2 Kv	124	177	308 x 59 x 92(49)		1956
Letten	2 Fh	35	615	197 x 36 x 47(40)		1956
Ligga	2 Kv	160	131	253 x 63 x 115(62)	74	1954
Langstrommen	2 Kv	41	100	154 x 49 x 92(49)		1962
Messaure	2 Fv	240	282	407 x 61 x 94(48)		1963
Mockfjard	4 Fh	12	79	105 x 31 x 36(30)		1910
Norrfors	2 Fv	26	39			1926
Norrange	2 Kv	40	69	125 x 50 x 94(43)		1963
Parteboda	2 Fv	29	103	89 x 41 x 115(77)		1960
Porjus	9 Fh	160	190	440 x 39 x 66	3, 6, 8, 20, 51, 57, 173, 180	1914
Ramsele	3 Fv	150	259	295 x 46 x 105(56)	181, 198	1958
Stalon	1 Fv	100	653	157 x 52 x 79(39)		1962
Storfinnforsen	3 Fv	115	164	213 x 46 x 98(46)	181, 198	1953
Stornorrfors	3 Fv	375	244	407 x 61 x 94(48)	164, 182	1958
Saalsjo	2 Fv	106	635	126 x 45 x 96(48)		1962
Trangslet	3 Fv	240	466	194 x 46 x 97(52)		1959
Tasan	1 Fv	35	883	59 x 36 x 92(40)	140	1953
Torpshammar	1 Fv	55	422	118 x 46 x 82(48)		1943
Umluspen	2 Kv	88	110	241 x 52 x 92(42)		1957
Vargforsen	2 Fv	92	161	262 x 59 x 85(43)		1961

Country and Plant (1)	Turbine No. and Type (2)	Plant Capacity MW (3)	Static Head Ft. (4)	Powerhouse Chamber L x W x H (5)	Reference Number (Section D) (6)	Year Initial Oper. (7)
SWITZERLAND					25, 96	
Bermano		100			51	
Biasco						
Cavergno	2 Ph	55	1670	x 92 x 71	118, 144, 173, 180	1955
Erstfeld	3 Fv	173	1033	145 x 55 x 109(57)		
Fionnay (Dixence)	6 Ph	336	2860		129, 144, 173, 180	196-
Fionnay(Mauvoisin)	3 Fv	127.5	1550		144, 159, 172, 173, 180	1955
Gondo			1605		173, 180	
Goschenalp	4 Pv	140			178	
Grimsel II					144	
Handeck II	4 Pv	114	1520	260 x 55 x 84	46, 51, 57, 90, 93, 144, 159, 166, 173, 180	1952
Innertkirchen	5 Pv	210	2235	328 x 64 x 85	12-18, 51, 57, 63, 93, 121, 144, 159, 166, 173, 180	1943
Lavaz					57	
Lavey	3 Kv	76	130	192 x 66 x 103	26, 36, 49, 51, 60, 61, 63, 77, 159, 173, 180	1950
Les Clees	3	26			159, 167	1956
Luzzone						
Mieville	3 Ph	85	4750	195 x 50 x (58)	72, 126	
Oberaar (Grimsel)	1 Ph	40	1740		46, 114, 144, 159, 173, 175, 180	1953
Olivone						
Palue					51	
Peccia	2 Ph	47	1400	x 92 x (71)	118, 144, 159, 173, 180	1955
Pfaffensprung	6 Pv	28	2428	x 62 x 103(57)		
Rheinau					173, 180	
Robbia						
Sasselo (Calancasca)	2 Ph	20	1352	130 x 130 x	40, 136, 159	1951
Somvix						
Tarasp						
Verbano	4 Fv	100	973		51, 118, 144, 159, 173, 180	1954
Zeres						

UNITED STATES						
Fairfax Falls,(Vt.)	3	1.1	75	(Caved in - 1927)	71, 203	1904
Haas (Calif.)	2 Pv	128	2448	173 x 56 x 100(56)	186, 203	1958
Snoqualmie Falls	4 Ph	6	270	200 x 40 x (30)	1, 2, 4, 71, 137, 173, 180,	1899
(Wash.)	1 Fh	5	270		203	1905
Spaulding No. 1 (Calif.)	1 Fv	6.4	197	85 x 30 x 60(28)	203	1917
					96	
USSR						
Niva	4		242		51, 173, 180	1949
Sevang						
Sulak					51	
YUGOSLAVIA						
Doblar	3 Fv	33	168	129 x 46 x 128	131	1939
Glava Zete	2 Kv	5	70	96 x 39 x 97		1955
Jablanica	6 Fv	168	345	360 x 66 x 110	130, 179	1955
Jajce I	2 Fv	50	300	212 x 62 x 118		1957
Jajce II	3 Fv	27	175	177 x 56 x 112		1955
Mavrovo	4 Pv	150	1800	495 x 52 x 72		1957
Medjuvrsje	2 Kv	7	65	190 x 52 x 97		1955
Ovcar - Banja	2 Kv	7	73	190 x 52 x 97		1955
Plave	2 Kv	17	82	98 x 42 x 132	131	1940
Ras	2 Fv	7	535	116 x 24 x 50		1955
Vinodol	3 Ph	84	2165	319 x 62 x 64	159	1952
Vrla I	4 Pv	48	1122	225 x 45 x 79		1955

Snowny Mountains Scheme - Table of principal features - Station T1 underground, with transformers underground, separate high voltage cable tunnel - T2 also underground - Stations M6 and M7 proposed as underground plants - combined capacity of M6 and M7 about 1000 MW.

211. RECENT DEVELOPMENT OF SWEDISH WATER POWER DESIGN AND CONSTRUCTION, Tore Nilsson. Fifth World Power Conference 1956, Section H/12, Paper 60 p 1-16. General characteristics of Swedish Plants - where head exceeds 115 ft. almost invariably cheaper to build underground power station - table of statistics of 29 underground plants existing or under construction - new practices which save cost such as draft tube above scroll case, improved crane design for reduced clearances.

212. TRENDS IN NORWEGIAN PRACTICE IN WATER POWER DEVELOPMENT, Ragnar Heggstad. Fifth World Power Conference 1956, Section H/21, Paper 109 p 1-14. Intensive building of hydroelectric power stations since 1956 - 56 new plants of which 24 underground - 26 underground plants existing with 12 under construction - advantages mostly economic - good results with "smooth blasting" - in all new plants transformers, and in many cases high voltage circuit breakers, underground - prevention of ice in air intakes important.

213. GLEN SHIRA HYDRO-ELECTRIC PROJECT, J. Paton. Institution of Civil Engineers Proceedings 1956.

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ARCH DAMS: DEVELOPMENT OF MODEL RESEARCHES IN ITALY

Guido Oberti,* M. ASCE
(Proc. Paper 1351)

FOREWORD

This paper is one of a group presented at the ASCE Symposium on Arch Dams, June, 1956, at Knoxville, Tennessee.

Since the last symposium on masonry dams (April, 1939), much progress has been made in the design and construction of arch dams and their appurtenances. This Symposium was planned to enable engineers concerned with arch dams to exchange their ideas and experiences for the benefit of all. The papers in this Symposium are "Arch Dams: Their Philosophy," by Andre Coyne (Proc. Paper 959); "Arch Dams: Trial Load Studies for Hungry Horse Dam," by R. E. Glover and Merlin D. Copen (Proc. Paper 960); "Arch Dams: Portuguese Experience with Overflow Arch Dams," by A. C. Xerez (Proc. Paper 990); "Arch Dams: Theory, Methods, and Details of Joint Grouting," by A. Warren Simonds (Proc. Paper 991); "Arch Dams: Santa Giustina Single-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 992); "Arch Dams: Measurements and Studies on Santa Giustina Dam," by Claudio Marcello (Proc. Paper 993); "Arch Dams: The Reno Di Lei Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 994); "Arch Dams: Isolato Double-Curvature Arch Dam," by Claudio Marcello (Proc. Paper 995); "Arch Dams: Rio Freddo Dam with Gravity Abutments and Cut-Offs," by Claudio Marcello (Proc. Paper 996); "Arch Dams: Design and Observation of Arch Dams in Portugal," by M. Rocha, J. Laginha Serafim, and A. F. da Silveira (Proc. Paper 997); "Arch Dams: Development in Italy," by Carlo Semenza (Proc. Paper 1017); "Arch Dams: Design of the Kamishiiba Arch Dam," by C. C. Bonin and H. W. Stuber (Proc. Paper 1018); "Arch Dams: Observed Behavior of Several Italian Arch Dams," by Dino Tonini (Proc. Paper 1134); "Arch Dams: Measurements and Studies of Behavior of Kamishiiba Dam," by H. Kimishima and C. C. Bonin (Proc. Paper 1182); "Arch Dams: Construction of the Kamishiiba Arch Dam," by K. M. Mathisen and C. C. Bonin (Proc. Paper 1183); "Arch Dams: Review of Experience," by Robert E. Glover (Proc. Paper 1217); "Arch Dams: Stress Studies for Ross and Diablo Dams," by Joe T. Richardson and Owen J. Olsen (Proc. Paper 1267); "Arch Dams: Economy

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of Concrete Dams," by Louis G. Puls (Proc. Paper 1286); Arch Dams: Development of Model Researches in Italy," by Guido Oberti (Proc. Paper 1351).

ABSTRACT

Experimental researches on models in Italy have greatly contributed to the national design of arch dams. Actual possibilities of models based on the theory of similitude are discussed. Important cases of Italian arch dams studied by models are described and principal results obtained are reported.

PRELIMINARY CONSIDERATIONS ON STRUCTURAL MODELS

Introduction

Research on models in the quiet atmosphere of a laboratory permits an accurate analysis of variables, and renders the experimental investigation simpler and less costly than on the prototype. Furthermore, it permits investigations of the static and dynamic behavior of the structure, predicting its real degree of security and realizing the best economy.(9)

The use of models is especially valuable when the mathematical solution of a problem is either unknown, extremely laborious, or difficult to define as to the boundary conditions.

The theory of models is based on a well-known principle of similitude which states that two systems are physically similar when there exists a geometrical correspondence between the points of the two systems, and the quantities of the same physical nature have a constant ratio at corresponding points. Complete physical similitude between prototype and model is reached when all the relations between the "scales" with which the model reproduces the physical quantities, on which the problem depends, are taken into consideration.

If there are n physical quantities upon which the problem depends and we choose the q functions which are fundamentally and dimensionally independent, corresponding to the degree of dimensional freedom of the problem (3 for mechanical problems), it is always possible to get $n - q = m$ dimensionless π ratios, which correspond to the m quantities relating each of these and the q functions assumed as fundamental. Having then chosen the dimensionless π_1 ratio relative to the quantity that one particularly wishes to know, this ratio becomes then the function of the remaining $m - 1$ dimensionless ratios $\pi_2 \ldots\ldots\ldots \pi_m$. The model will therefore correspond to the prototype in all respects if the values of these ratios remain unchanged in passing from the prototype to the model, and the π_1' ratio for the model will then also be $\pi_1'/\pi_1 = 1$; a relation which permits, having measured the quantities on the model, to obtain the corresponding value on the prototype.

In addition, the inherently dimensionless physical constants upon which the problem may depend must be conserved in passing from the prototype to the model, for example: Poisson's ratio, the coefficients of friction between the various materials, and so on.

It is useful to distinguish between the cases that one may wish to study on

a model, separating those for which one possesses a thorough mathematical theory, from those in which this is not so. A classical example of the first case is presented by a study of the behavior of any structure made in a homogeneous isotropic elastic material and bounded in a statically determinate way, as mathematics then offers the theoretically complete solution to the problem: a system of differential equations having as unknown functions the stresses.

Even though the numerical solution of such a system is often extremely laborious, as for example in the most complex problems of concrete dams, the knowledge of the theory simplifies the investigation with the model as the equations of the theory furnish a complete and precise list of the quantities which influence the phenomena studied. In particular, it should be remembered that such equations postulate the independence of the stresses from the physical-mechanical characteristics of the material (elastic modulus) yield point (with the exception of Poisson's ratios for three dimensional problems) therefore permits the use of model materials which differ from those of the prototype, as applied in photo-elasticity.

Without a theory and the relative equations which set out the physical problem to be studied, it is more difficult to realize a complete similitude, as naturally all the non-dimensional ratios on which the phenomenon we study depends, may not be identified. It is in these cases that dimensional analysis, as a precious resource, becomes of use, in fact, once the quantities which are present in the phenomenon are listed, it provides us with the independent dimensionless ratios which can be built with these quantities and thereby provides a guide to the proper use of the model to obtain satisfactory results.

It seems advisable to mention these preliminary fundamentals of the model theory in order to outline the difficulties that beset work on models. Such difficulties are often present in hydraulic, electric and aerodynamic researches.

The Structural Models

a) Structural investigations generally present favourable conditions since the independent fundamental quantities upon which the behaviour of a structure depends are generally three; the classical "length," "mass" and "time," or three equivalent dimensionally independent quantities and that these are reduced to two when only the static behaviour of the structure is studied, as in this case the variable "time" is missing.

If λ and x represent respectively the ration of similitude of the lengths and of the forces, the ratio of similitude ζ between the stress values, during the passage from the prototype to the model must be:

$$\frac{\chi}{\lambda^2} = \zeta \qquad (1)$$

All the other physical quantities, occurring in the problem, which have the dimensions of a stress (modulus of elasticity, yielding value, failure unit loads) must then have this same ratio. The materials which the models and their foundations are built, must generally conform to this same ratio which we will call "effectiveness ratio." (In the elasticity problems only and within

the limits of the mentioned theory, this dependency may be avoided. For example, photo-elasticity utilizes materials for models which are quite different from those of the prototype.)

In the particular case of only superficial loads, with π indicating the ratio between the intensity of these loads the required relationship will be: $x = \pi\lambda^2$ and here π coincides with ζ, the latter is then independent of the scale ratio λ.

But if the stresses due to dead weight are not negligible, and ρ represents the ratio of the densities, then it will also be necessary that

$$x = \rho\lambda^3 \tag{2}$$

and therefore the condition that is obtained by placing (2) into (1) must be considered. This is

$$\zeta = \rho\lambda \tag{3}$$

The difficulties are increased by this requirement which may justify the expedient of using large scale models and also to increase—with artificial devices—the density of the model material.

In the particular case where the most important stresses are due to body forces, as for example, in dam problems (hydrostatic load and dead weight) only the relationship (3) is required.

When it is possible to find materials to build a model and its foundations for which the conditions of invariability of ζ are met, in the sense that the "intrinsic curve" of the model-material is similar to that of the prototype in the constant ratio ζ and the scale of the density ratios satisfy the relation (3), similitude may be considered attained and it may be then considered effective not only within the limits of elasticity but as far as to the breaking point.

This result, which the author reached some twenty years ago, permitted, with the use of convenient materials, to study on models with tests up to the breaking point, the degree of safety of important structures and in particular that of practically all the big Italian dams designed and built in the last decade.

For several years plaster of Paris had been employed, but it was rather difficult to use, specially when the models were large and thick, because the plaster took a long time to dry and the interior was not uniform. That was the principal reason which led to the use of a special mixture for the most recent models. It was a special concrete in which the aggregates were volcanic pumice stone from the island of Lipari. Such stone is cheap in Italy and is therefore very useful for laboratory tests. Using volcanic stone, tests had been made with different percentages of cement, and it is now possible to construct a model of this special mixture having a large variation in the ratio of its modulus of elasticity to the modulus of elasticity of the concrete of the dam.[11] In some cases the ratio is as low as one to twenty. Furthermore, in the large models it has been possible to include the natural rocks in the model, incorporating the same ratio as existed in nature. This was done by means of research made in the field by means of special devices, particularly for limestone. The necessary information having been gained by measuring

"in situ" the deflections (in all directions) caused by pressure in a large tunnel which had been excavated in the interior of the mountain.

When one has to undertake experimentation on models of this kind it is convenient to begin the research work directly on the strains ϵ, assuming them as fundamental unknown quantities (instead of stress components); these being already dimensionless will insure that a complete similitude will be obtained when they are equal at the corresponding points of the prototype and of the models. All the precedent considerations of the case are still valid and, in particular, the relation (3).

Then the strains on the model, which it is possible to measure with a high degree of accuracy, by the use of extensometers having a very high magnification (fig. 1) will be equal to those of the prototype.

It is thus learned that the displacements and in particular the deflections (that dimensionally are $\epsilon \cdot \iota$), will be proportional to the scale ratio λ between the prototype and the model (fig. 2).

b) Previously when static problems alone were considered, deformations and stresses were not influenced by "time."

In reality the collapse of a structure may also depend, even if in a different degree, on the time of the application of loads because of the viscosity of the materials of which they are built.

It is known from theory that, in such cases, the stresses are linearly related to the corresponding velocity of deformation through a coefficient of viscosity, which is a constant providing the material is homogeneous and isotropic. It is then necessary to add a new fundamental quantity, such as "time," for which the non-dimensional ratio μ between the viscosity coefficients must be equal to $x\lambda^{-2}\tau = x\zeta$ with τ representing the ratio of times between the prototype and the model.

The variable "time" comes into play again when the dynamic behaviour of a structure must be studied. This is of particular interest for seismic effects. In such cases it is necessary to bear in mind that among the forces to be considered in the first place are those of gravity, and being unable, obviously, to alter the value of the gravity acceleration in passing from the model to the prototype, one is obliged to presume that this quantity is a fixed dimensional constant. It is useful therefore to consider acceleration as a fundamental quantity to add (instead of time) to the two preceding ones and the ratio between the accelerations acting on the prototype and in the model must be equal to one. The ratio of the times τ must then satisfy the condition:

$$\zeta = \sqrt{\lambda} \tag{4}$$

It follows that the vibrations will reproduce themselves on the model with a higher frequency. For example, on a model having a scale of 1:100 the frequencies will be ten times as high as in the prototype (Froude-similitude).

It is useful to point out that in order to satisfy relation (4) it is not possible to work a model made of the same material as the prototype (or which has the same density) since, said ρ the ratio between the density of the prototype and model materials, must be in any case (the forces being dimensionally equal to the product of mass times acceleration):

$$\zeta = \lambda \rho^{\frac{1}{2}} \zeta^{-\frac{1}{2}} \tag{5}$$

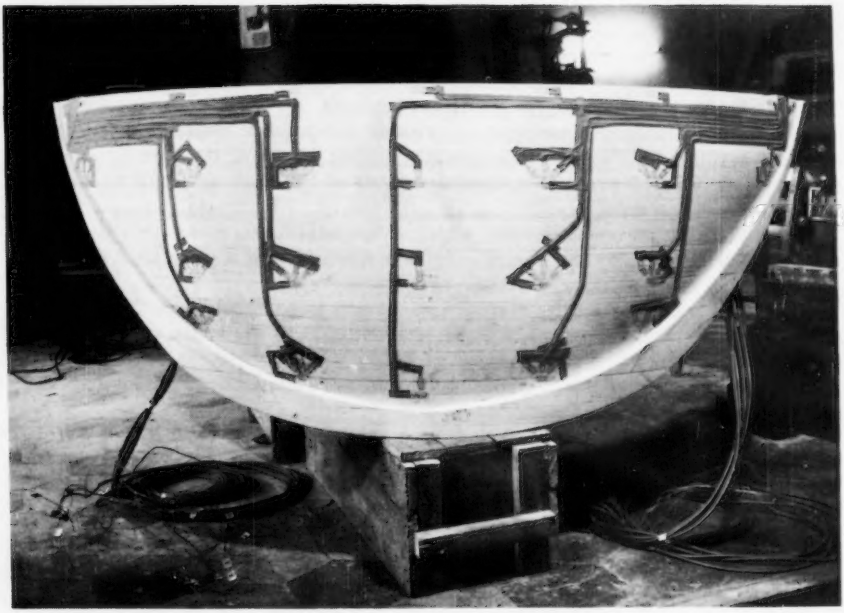

Fig. 1. Electrical Strain Gage Equipment in Place on the Upstream Face of an Arch Dam Model.

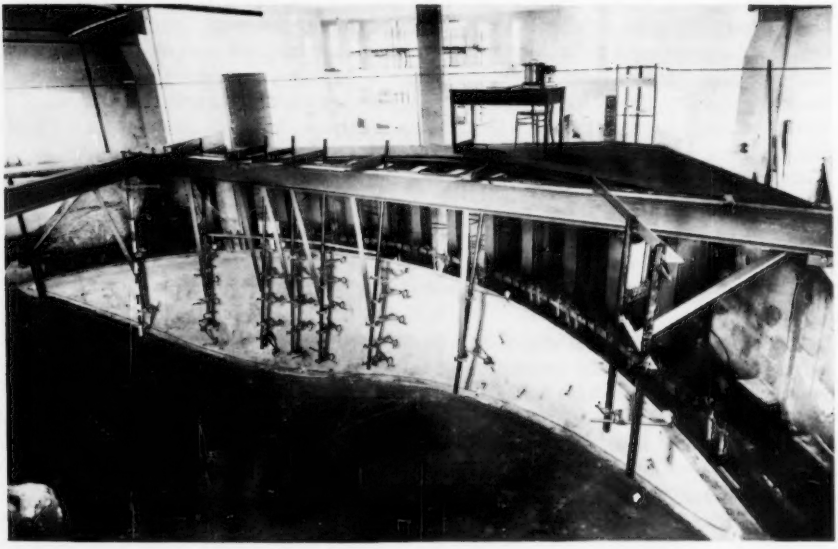

Fig. 2. Application of Dial Gages on the Downstream Surface of a Large Arch Dam Model.

The equations (4) and (5) are satisfied by imposing the condition (3).

As an example, in the case of the model studies of the earthquake-effects on the Ambiesta dome dam, by conforming to the requirements of these fundamental ratios, we have assumed

$$\lambda = 75 \qquad \zeta = 50 \qquad \rho = 3/3$$

using for the model a special mortar of litharge and plaster.

c. With regards to what is assumed in order to obtain by calculations and by direct research conducted on the real dam, models have been objected to on the grounds that internal stresses are not examined and that thermal effects are not studied.

For the first question it is pertinent to point out that—as a rule—the stresses of more interest are found on the surface. Presently, internal measurements at single points in models are possible with a good accuracy.(12) Furthermore, if one limits himself to the elastic behaviour of plane structures, it is also possible to obtain by photo-elasticity, the stresses in the interior.

With regards to the second point, it may be interesting to remember that, should the thermal variations (or its equivalent as shrinkage) not be directly modelled but are linear functions of the coordinates, the thermo-elastic problem may be replaced for elastic conditions, by the artifice of applying on the model suitable volume or surface forces instead of the thermal actions (and with the precaution to measure also the deformations, which the model, being free from redundant ties would develop under the action of such ideal forces.)

Generally speaking, if the deforming actions are deduced to simple Volterra dislocations, it is possible to employ models. Only if one passes to more general types of distortions, does the use of models become really arduous as it is then necessary to reproduce and to model the local physical causes which introduce such distortions.

The Evolution of Experimental Methods on Models

In Italy the experimental procedures on models have been progressively extended in these last years and I would divide them, following their chronological evolution, into three groups of substantially distinct methods.

a) With the methods used in the first group the plain elastic problems are studied: prevailingly with the aid of photo-elasticity and deformeters. Photo-elasticity is a particularly elegant system which permits us to obtain photographic reproductions of stress patterns, lines of uniform shear stress, and to deduce the stress-trajectories (isostatic lines) in the prototype by the experimental observation of the isoclinics obtained in the model. This method of research is now successfully used in many laboratories which deal with structures.

The deformeters, developed from the first Beggs types, are also tools of great utility.

b) The methods of the second group investigate three-dimensional elastic problems with extensometer measurements; that is by applying directly to the model mechanical, optical or electrical extensometers.

Many structures were thus studied in Italy (especially by the author in the

laboratory of testing models in the Polytechnical School of Milan) by adopting model materials (celluloid, plexiglas, etc. . . .) which were very different from those of the prototype provided that they are elastic. In these tests the continuity of the structure, the rigidity of the abutments or the elastic deformability of the ties were accounted for. In arch dam models the load was applied usually with quick-silver (fig. 3).

Under this condition the model functions as a "mechanical calculator" of stresses and gives results that may be usefully compared with those of the various calculations made for the study of the elastic behaviour of the structure.

c) Lastly, with the third group, having ascertained that some structures, particularly those of concrete, do not conform to the postulates of elastic theory, and that the noncompliance gives better results, it appears preferable to search for a more exact similitude with the prototype rather than for confirmation of elastic calculations. This is a decisive step towards conformity with nature which characterizes the considerable amount of work done by the ISMES in Bergamo in building models, which tend to substantially reproduce not only the behaviour of materials but also the particulars of the form and the performance of the prototype, of the bearing seats and the real deformability of the ties and of the foundations.

The tests made on the model may then be divided into two distinct and successive steps. In the first series of tests which we call "normal load tests" deformations are measured in order to obtain the values nearest to the condition of similitude, which will impose equality of the strains on the prototype and on the model, under normal load conditions corresponding to those of the structure in service. It is important to point out that during the application of the load various types of inelastic adjustments or permanent sets may take place which is good to stimulate, by repeating load cycles, reaching a regime working of the model which will be elastic, regular and suitable for the measurements of deformations and for useful controls. It is then possible to evaluate the stresses (knowing the stress-strain diagram of the material) and the static behaviour foreseeable in the prototype during normal service.

It must be observed that the stresses thus observed may not agree with those deduced, by calculations, since the adjustments (which generally have a beneficial effect) are not taken into consideration.

Having finished these tests and the relative measurements one passes gradually to the destruction tests or ultimate load tests. It is convenient then to assume as the overall safety coefficient of the structure, the ratio K_s between the value of the maximum load actually supported and that considered normal during service.

In the particular case of arch dams, having made the ultimate load tests on the model (conducted until collapse or until the first signs of cracks appeared on the upstream face), the safety factor will simply be the ratio between the maximum final value γ'_m of the specific weight of the liquid, fictitious or real, acting on the model, and the value γ' relative to the liquid conforming to the foreseen "normal load" (which is realized in the dam for the designed maximum water level). If ζ is the "Ratio of Effectiveness" between the material of the dam and that of the model, λ is the scale ratio, $\gamma_o = 1\ T/m^3$ is the specific weight of water acting on the prototype, the following relations will hold:

Fig. 3. Model of Arch Dam Tested with Mercury in a Rubber Tank Acting on Upstream Surface.

$$\gamma' = \frac{\lambda}{\zeta}\gamma_0 \quad \text{and:} \quad K_s = \frac{\gamma_m'}{\gamma_0}\frac{\zeta}{\lambda} \tag{6}$$

In the case of gravity or arch-gravity type dams in which the effect of weight (also a volume load) has the essential function of maintaining stability, it is necessary to arrange the model for its increase and to maintain it at the correct ratio in respect to the hydrostatic load (Fig. 6). It is with this precaution that we may proceed with the ultimate load tests. If in these tests a safety ratio, which is considered sufficient under normal conditions, is reached without collapse of the model, it may be interesting to continue increasing only the hydrostatic thrust and not the weight in order to learn the strength at exceptional thrusts (such as earthquakes, bomb effects, and so on).

These latter methods in our opinion represent a notable progress in comparison with the former which, on the other hand, are still useful especially as a means of comparison with the theoretical results. In fact the methods of the first two groups are based, on a set of hypothesis, that lead to results which may not conform to reality since they would only hold for an ideal structure which faithfully obeys the initial hypotheses.

Instead, our methods, rather than obeying preconceived idealizations come closer to the reality of a specific case. Thus we do not hesitate to introduce into the model materials, foundations, ties and joints and the general building particulars, which may prohibit the possibility of an analytical check (and producing sometimes a certain dispersion of results) but, compensate by a better agreement with the real case and therefore adhere closer on the true and final aim of the designer.

This point of view permits and justifies the use, in these third methods, of useful devices which may produce local troubles which are negligible in respect to the final aim of the tests. Thus, for instance, the hydrostatic load on the up-stream face of the dam models may be applied with hydraulic jacks provided with suitable diffusion plates instead of liquids; also the corrective addition to the dead weight may be concentrated in a number of points instead of being distributed continuously on the whole volume of the structure. With similar practical means (the influence of which is possible to check experimentally) the conditions of similitude are retained in the model and problems, which otherwise would be difficult or unsurmountable, are resolved.

This conclusive phase of experimentation which is more comprehensive and delicate than the preceding ones, requires a critical attitude of mind, a good experimental ability, a patient research on, and preparation of materials suitable for the construction of models, and coatings capable of preserving them, specially from shrinkage, during the tests. This is what has been developed and studied in these last years in the ISMES laboratories.

MODEL AND STRUCTURAL TESTING AT THE I.S.M.E.S.

1. In order to fully appreciate the function performed by the Instituto Sperimentale Modelli e Strutture, "I.S.M.E.S.," (Model and Structural Testing Institute) of Bergamo (Lombardy Region), it should be noted that in Italy most of the study and research work in the field of civil engineering is done by University Institutes. Their work is largely theoretical, and in the field of

experimental research it is confined chiefly to material testing, due also to the limited facilities and staff available for this work. The I.S.M.E.S., instead, is a private corporation established for the purpose of solving, with adequate financial means and the freedom of action which, by their own structure, University Institutes do not possess, specific structural problems arising for designers and builders. Therefore, I.S.M.E.S. carries out a technical-scientific activity which is, so to say, complementary to the activity which is carried out, or should be developed, in the Italian University Research Laboratories.

I.S.M.E.S. was established by a group of companies and contractors including: EDISON Co. of Milan, ITALCEMENTI Co. of Bergamo, Acc. FALCK Co., SADE Co. of Venice, SIP Co. of Turin, "Societa Italiana Partecipazioni Industriali," SME Co. of Naples, MONTECATINI Co., ROMANA ELETTRICITA' Co., SELT-VALDARNO Co., TERNI Co., ACEA of Rome, AEM of Milan, AEM of Turin and the contractors: GIROLA, ITALSTRADE, LODIGIANI and TORNO. The scope of the Institute's work extends to experimental research on the behavior of structures, by means of tests conducted on large three-dimensional models, or on the structures themselves at the construction site.

The experimental study of the structures by means of models has been gaining momentum in the last few years, as a result of the improvement of measuring instruments, and has gained increasingly wide acceptance as an effective aid by open-minded designers and builders.

The practical usefulness comes from the fact that it is possible to work out a solution for complicated structural problems even in the cases where calculations cannot provide sufficient assistance.

This process yields, in the design stage, valuable information making it possible to anticipate the static or dynamic behaviour of the structure and, if necessary, to select among several designs the solution which is likely to produce the highest efficiency and the lowest construction cost. In addition to tests under normal load conditions, the Institute usually carries out ultimate load tests intended to yield an indication as to the order of magnitude of the overall safety-coefficient of the structure modelled.

So much for what we can define as the "technical" work which the Institute is called upon to do for its customers in Italy and abroad.

In the field of pure scientific research the Institute studies improvements of the techniques for research on structural models. In this field, some results of fundamental value have already been achieved, such as the extension of model testing beyond the elastic limit, thanks also to the development of suitable materials and measuring instruments.

The Institute officially was established in 1951, and is in a phase of continuous development. Among its equipment are special structures of heavily reinforced concrete, built to contain large models or structural elements to be put through static tests, and to support without appreciable deformations the loads involved in the testing.

One of these structures is a rectangular-base tank measuring approx. 32 x 16 feet, particularly suitable for testing model dams; and the other is a circular-section tower, 32 feet interior diameter and 60 feet high, designed to hold tall models (dams built in narrow gorges, skyscrapers, cement silos, etc.). The tests on models or structural elements exerting no thrusts above ground level, i.e., resting or anchored upon level ground (penstocks, floors, etc.) are conducted in a large shed-type building.

A new department for research on the physical-mechanical characteristics of concretes, including those with large-size gravel, has been activated last year. It is equipped with a 2,000-ton compression and bending materials-testing machine, which can also be used for studies on various structural elements (pillars, girders, etc.)

The Institute possesses an extensive set of loading devices (hydraulic jacks, springs, etc.) to apply on model or structures any kind of stress, such as dead loads corresponding to their own weight; accidental loads, hydrostatic loads, wind pressure, etc. Its laboratories are also equipped with a complete set of measuring devices (bending gauges, stress gauges, recording units, etc.) and with auxiliary equipment (including a photo-elastic section for research in two-dimensional elastic fields) which are used, in specific problems, to supplement the research on three-dimensional models.

A set of loading and measurement devices was developed for field tests. Particularly interesting is the equipment for the determination of the deformability of foundation rock by underground tests (special jacks, pumps, waterproof gauges), which is one of the fields in which the Institute specializes.

The Institute has ready for use from this year a set of equipment, the only one of its kind in Europe, which is used for studying on models the effects of seismic (or, generally, vibrating) actions on building structures and dams. The seismic testing devices include:

a) A metal platform, measuring approx. 10 x 15 feet, of highly rigid construction, upon which will rest the structure to be tested, complete with foundations (Fig. 6);

b) Three vibration-generating units, including:
   1. A variable-constant pendulum and spring system for the reproduction of shock waves;
   2. A centrifugal vibrodyne for the reproduction of unidirectional harmonic movements, capable of applying forces up to 10 metric tons;
   3. Four electronic-controlled electro-magnetic vibrators, for the reproduction of both unidirectional and vortex-type movements;

c) An equipment for recording the strains and distortions of the model during the testing.

Better than any list of equipment, however, the capabilities of the Institute can be illustrated by the record of research work done.

2. Systematical tests on models, including some of very large size, have been conducted by the Institute, particularly for large dams.

The models of dams are built and tested in accordance with specifications which are the product of long experience, and therefore usually applied to different models. The following is a brief description of the processes and specifications usually adopted for arch dam tests.

The first step is the construction of a preliminary model, of plaster or wood, which is used to study the design details required and for the construction of molds for casting the actual model. When necessary, this preliminary model is also used, on completion of tests, to reproduce the principal-stress trajectories.

The actual working model is laid on a foundation bed of the required "modulus," and its construction follows quite closely actual prototype construction processes. Among other details reproduced are the joints to be grouted by injection when the setting process is completed (Fig. 4). Concrete

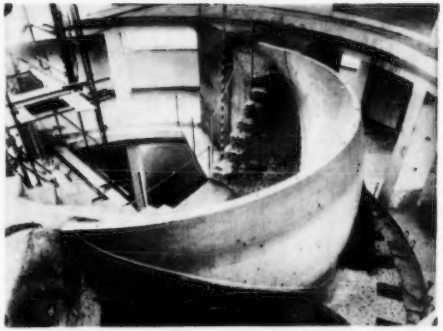

Fig. 4. The Model (1:50) of the Beauregard Arch-Gravity Dam Immediately after Casting. The Holes for the Injection of the Joints May be Seen.

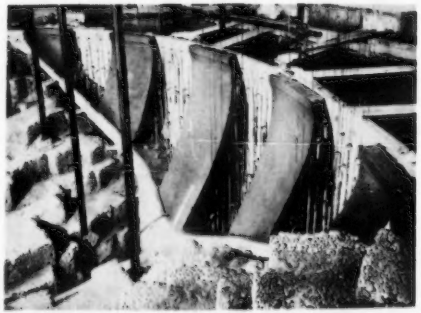

Fig. 5. Casting a Model Arch-Gravity Dam; Tie Rods for the Application of the Dead Load are Seen.

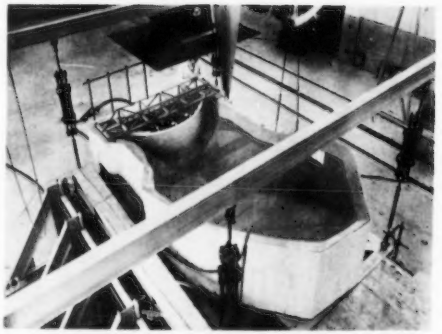

Fig. 6. Vibrating Table for Earthquake Tests. Model (1:75) of the Ambiesta Dome-Type Dam Ready for Seismic Tests.

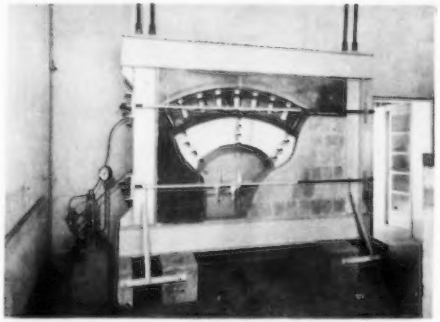

Fig. 7. A Hydrostatic Load Test, by Means of Jacks, on a Single Arch Model with Abutments on Two Different Kinds of Rock (Modulus Ratio 1:10): Preliminary Tests for Beauregard Dam.

is reproduced by suitably dosed cement and stone pumice mix, approximating the mechanical characteristics of the actual rock and construction materials.

Before pouring, fastenings for additional own-weight loads are inserted at appropriate points in the body of the models (Fig. 5). On completion of the hardening process, vertical loads are applied by means of spring-loaded dinamometers (Fig. 6). Water pressures are usually reproduced by means of hydraulic jacks, fitted with special attachments which distribute the load over a sufficiently wide surface. In elastic tests, local measurements are taken by means of extension gauges of various types when spot readings are possible or by centralised electrical indicating systems. Mechanical dials (deflectometers) are also used to measure bending deflections and overall shifts in the structure.

On completion of the normal tests, which lead to the determination of the stresses and, as a rule, to the tracing of the stress trajectories on the model structures, destruction (ultimate load) tests are conducted. The loads are gradually increased until the model collapses, yielding the value of the overall safety coefficient.

In the individual practical cases, the technical and financial importance of this research has always proved substantial. As for the financial side of the problem, we would like to mention here as an example the fact that the results of tests conducted with the first model built (Pieve di Cadore Dam) have led to reducing the overall volume of the structure by about 15%, with a saving in the order of $1,600,000.

In some cases the scope of the experimental research was extended to include a comparative study of different alternate solutions. This was done, for instance, in the case of Osiglietta arch dam (acc. Falck Co.) and more recently for the SADE Company's Fedaia Dam, when model tests were conducted on two different designs of arch-gravity dams, and on a third (then actually adopted) for a buttress dam.

In other cases, model tests were conducted for the purpose of observing the influence of particular conditions on the static behavior of the structure. Thus, for instance, in the case of SIP's Beauregard Dam, the Institute's research staff has reproduced, after overcoming considerable difficulties, the pronounced difference in elasticity of the two valley sides (with elasticity coefficients in the ratio of 1:10) (Fig. 7). In the case of the Cancano Dam (AEM), adjustments were made to allow for the heterogeneous structure of part of the rock on the right abutment, reproducing the precise lay of the rock layers.

For the Val Gallina dome dam and for the arch-gravity Piave River Dam, a comparison has been conducted between the deflections observed on the model and that of the actual structure, and the results were found to substantially agree.

For the Giovaretto buttress dam, a study has been conducted, by means of model, of the stresses in the highest spur, extending it to the inner part around a lightening and drainage hole, around which the existence of tension stresses was feared. Studies of the same type were conducted on the arch-gravity Cancano Dam mentioned above as regards the concentration of stresses around some large tunnels and the elevator shaft.

For the Falck Steel Company, studies have been conducted on three models representing the arch-gravity Frera di Belviso Dam, which is designed to be built in two separate stages (Fig. 8). Because of the particular shape of the joint between the successive construction stages, the static behavior of the

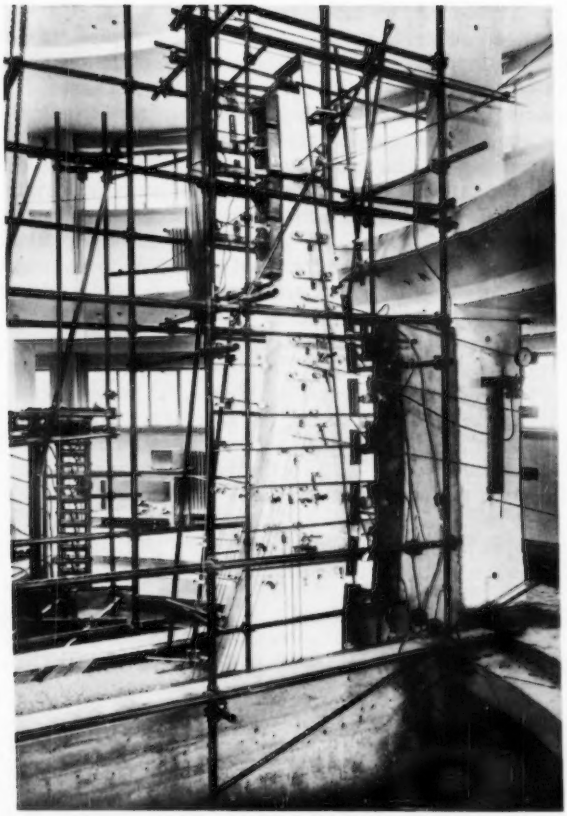

Fig. 8. Testing of a Single Cantilever Model of a High Arch Dam Designed for Construction in Two Steps with a Longitudinal Joint.

completed dam involved problems which could be studied only on an experimental basis.

Outside the sector of dams, different structures and structural components have been investigated. For instance, studies were conducted into the actual pushing action exerted by pulverulent materials (cement) in tall storage silos. Researches were carried out, by laboratory and site tests, on the static behavior of pre-stressed concrete penstock and acqueduct pipe, in different practical cases involving pressures from 50 to 350 tons per square meter. The Instituto has already completed the experimental tests on a 1:15 scale model of the skyscraper which will be built in Milan for Pirelli Co. to a height of 450 feet above foundation level. The reinforced concrete structure was modeled in its essential lines, basically respecting cross-sections, moments of inertia of the individual parts and the rigidity of the joints. The model was built of a pumice-cement mixture reinforced with iron wire and mesh. For the foundation slab, the pre-stressing framing was also reproduced. Particularly interesting were the wind-pressure tests, conducted by measurements under static load by recording of the dynamic effects. In addition to the stress pattern, these tests made it possible to ascertain periods and accelerations involved in connection with bending and torsion oscillations.

The basic of this brief and incomplete review of the activities of I.S.M.E.S. is to show, in the light of facts, that testing structural models is not only useful towards the distant goals of progress, but also serves the cause of immediate economy. It is worth recalling—as written by Prof. Danusso, the Institute's President—that we are still forced to use but a small fraction of the actual resistance of the construction, since we must provide a wide margin of safety against our own ignorance. This margin can be largely reduced by using models, which come much closer to reality than any calculation, and therefore can give much better answers for many construction problems, not only in arch dams.

TESTS ON ARCH DAM MODELS AND THEIR RESULTS

Two-Dimensional Models—Photo-Elastic Tests

Early tests (1935-36) were carried out employing photo-elasticity as a means for investigating the problem of the elastic behaviour of the arches of dams having a heavy thickness, which was particularly interesting for the designing of a large pure arch dam, that of Santa Giustina (Edison group Co.).

The arch was assumed to be elastic, continuous, and set on a rock yielding elastically with the same modulus as that of the dam material. Many arches were investigated with a circular upstream face outline, both at constant and variable thickness and with various openings, from about 60° to about 120°.(1)

From the analysis of the results and their comparison with those of the ordinary elastic calculation, it was possible to draw the following general conclusions:

a) The elastic calculation gives results which are close to the experimental ones for arches having average slenderness ratios (radius/thickness) higher than 3, and having notable central angles. Decreasing such central angles (arches narrower than 90°), it was observed that the calculation leads as a rule to higher tensile stresses at the abutment extrados, slightly lower, instead, at the intrados, both at the crown and at the

abutment, with a closer agreement the higher is the slenderness ratio.

b) For thick arches of limited angular opening, only tests carried out on models can give positive results. Neither the elastic calculation, nor the quite simple one based on the (fairly common) assumption of assimilating such structures to ring elements are even approximately approaching reality.

The investigation emphasized also the desirability of designing arches with variable thicknesses, particularly when dealing with arches having small slenderness ratios and reduced openings.

The above considerations, although concerning facts today accepted everywhere, represented at the time of testing a substantial contribution to the designing of such high dams. These results, as well as other test results which will be briefly discussed further on, are here referred to essentially for their historical interest.

Three-Dimensional Models of Thin Arch Dams

a) Early investigations concerned a 70 m. high single curvature arch dam: the Rocchetta Dam, erected (by Falck Steel Co.) in 1937-38. The dam, set up directly against the rock banks, has angular openings varying from 100° at the crown to 120° at the bottom; thicknesses at the middle section varying linearly from 3.15 m. at the top, to 11 m. at the bottom.

The model (scale 1:40), besides being tested under normal hydrostatic load, in elastic conditions, was also subjected to endurance tests, up to collapse. The tests led to favourable conclusions about the static behaviour of the Rocchetta Dam and threw light on the manner in which this type of structure actually reacts to the water loads applied to it, and more particularly to check experimentally for the first time the great safety resources of arch dams. The following general conclusions were thus drawn for arch dams of a type similar to the one tested:(3)

1. If the structure has a high safety factor, as is the case with the Rocchetta Dam, a substantial contribution to strength exists, offered by the cantilevers, actually acting more like beams fixed at the bottom and elastically supported by the crown arch; such contribution slightly relieves the stresses in the horizontal arches.

2. The "ultimate load" tests showed that if the structure has not too high a safety factor, which would be the case if the ultimate strength of the concrete were low, or if the structure, remaining geometrically similar to the one under test, were much higher, it tends to work as by independent arches, disengaging from the cantilever effect. The presence of joints, the unavoidable slight settlement at the abutments, the plasticizing of the material, all favour the deflection of the arches, which, if heavily loaded, tend to work almost as individual arch-rings.

3. The usual calculation, which considered only independent and clamped arches, associated with the high safety factors imposed in Italy, did not agree with the static behaviour of the normal structure; it could show a better correlation, and limitedly to the superior part of the dam, if the safety factors of concrete were reduced.

4. The assumption made to consider the inferior horizontal section of the dam as comparable to ring elements uniformly loaded at the extrados does not agree with reality.

The analysis of the results obtained in this first investigation supports what had already appeared in the early tests on the Stevenson Creek Dam models about the considerable flexural stresses, acting on the cantilevers.

Keeping in mind also the balancing effect of their own weight, the idea arose quite spontaneously to no longer consider single curvature dams, but double curvature ones. The chance presented itself shortly after when, on behalf of the same Company, we were to study on a model the Osiglietta Dam to be built across a mountain gorge of about 215 m. span and 75 m. height, therefore very open and, in those days at least, exceptional for an arch dam. In order to compare the two types, the conventional, single curvature one and the double curvature one (dome shaped), it was decided to make parallel tests on models of the two types. In consideration of the character of the investigation, it was deemed convenient to use celluloid models and to apply the hydraulic loading with mercury. The tests were therefore limited to the elastic range.

The results obtained were extremely interesting and confirmed that the adoption of the second curvature did considerably reduce the tensile stresses along the cantilevers, improving the strength contribution of such vertical elements, from the fact that they acquire an effective curved shape. It was also observed that, in the specific case, given the high span-height ratio and the slenderness of the structure, it was convenient to increase the thickness of the crown arch. But another interesting fact emerged from the experimental study: the possibility of reducing the flexural stresses along the perimetral abutment of the structure caused by the clamping into the foundation rock, assumed here—as it was customary in those days—practically indeformable. Such a clamping effect is particularly harmful since it has a strong tendency to induce tensile stresses along the upstream face, as was clearly shown by the mentioned early tests on the models of the Rocchetta and the Stevenson Creek Dam.

The idea then occurred of radically modifying the tie condition, no longer by clamping the dam into the rock, but terminating it with a perimetral continuous joint against a concrete support saddle ("pulvino") solidly fixed to the rock. Tests carried out on a model with a perimetral joint fully confirmed the anticipated views, and quite safe stress values were obtained also on the downstream face.

A problem similar to the one above, rose with the Val Gallina Dam of S.A.D.E. which closed a large gorge having a somewhat similar shape as the Osiglietta Dam, but of larger size: 90 m. high and 240 m. span. An early model of this dam had been investigated at the Polytechnic Laboratory Structures of Milan (1947), but the results then obtained were later found to be partially changed by the variations introduced in the design of the dam at the time of construction as well as by the findings of further geological research which had emphasized the peculiar non-uniform condition of the dolomite foundation rock. For this reason, a second model was tested later, at the I.S.M.E.S. laboratory, on a 1:100 scale.(8)

The foundation rock for the whole of the volume which might have affected the stresses of the dam, i.e. for about 30 m. upstream, 60 m. downstream, and 40 m. deep, was then reproduced with the model. This zone was

subdivided into single sections, by means of watertight thin layers, so as to permit the variation of the deformability of each section in agreement with the researches made at the site.

The model dam—cast in a pumice-cement mix according to the most advanced technique—reproduced all the alternations brought to the design during the construction of the dam; particularly, the increase of the thickness of the right side of the dam-abutment zone and the realization of the perimetral joint, which was complete along the sides and limited to about one-third of the thickness upstream at the central part of the dam.

From the elastic tests it was also possible to draw the isostatic lines (stress-trajectories) which showed the characteristic trend already noticed in testing other similar types of dams; this trend should confirm the validity of the method of calculation based on inclined arches, along planes approximately perpendicular to the average fibre of the main cross section, rather than on horizontal arches.

The existence of isostatic lines that do not follow the traditional "horizontal arches-vertical cantilevers" lattice, is confirmed for every single or double curvature dam having a high span-height ratio, by the mentioned tests on the Rocchetta, Osiglia and Val Gallina dams, which cast doubt on the "trial-load" or "Ritter" type calculations, based only on the "horizontal arches-vertical cantilevers" lattice.

In this connection, a certain interest attaches to the results of an approximate theoretical investigation which were based on the measurements taken on the last model of the Val Gallina Dam.

For an evaluation of the water load distribution among the various analyzable strength capacities in the dam—evaluation in which, in ultimate analysis, the comparison with the calculation methods consists—assuming as coordinate axes the horizontal one, x, according with the parallels (arches), and the other, y, according with the meridians (cantilevers)—the behaviour of the dam was assimilated to that of a shell defined by the inter-actions (per unit length of principal normal sections of the shell): N_x, M_x, M_y, Q, M_{tx}, M_{ty} which—admitting the linearity of the law of variation of the stresses inside the structure, a characteristic proper of thin vaults—are obtained from the stress test values.

N_x normal force in the arch element; N_y normal force in the cantilever elements; M_x bending moment in the arch element; M_y bending moment in the cantilever element; Q shearing force in the cantilever; M_{tx} and M_{ty} twisting moments.

$$N_x = h \cdot \frac{\sigma_m + \sigma_v}{2} \qquad M_x = \frac{h^2}{6} \cdot \frac{\sigma_v - \sigma_m}{2} \qquad N_y = h \cdot \frac{\sigma'_m + \sigma_v'}{2}$$

$$M_y = \frac{h^2}{2} \cdot \frac{\sigma'_v - \sigma'_m}{2} \qquad T = h \cdot \frac{\tau_m + \tau_v}{2} \qquad M_t = \frac{h^2}{6} \cdot \frac{\tau_m - \tau_v}{2}$$

where h is the vault thickness, σ'_m and σ'_v are, respectively the normal up-stream and down-stream stresses in the cantilever direction, σ_m and σ_v are respectively the normal up-stream and down-stream stresses in the arch direction, τ_m and τ_v are respectively the tangential up-stream and down-stream stresses.

By indicating with p_m the water load referred to the mean fibre, r_1, and

$r_2 = \frac{r_x}{\operatorname{sen}\phi}$ the radiis of curvature of the shell, the equations of equilibrium takes the form (5):

$$P_m = \left(\frac{N_x}{r_2}\right) + \left(-\frac{1}{r_x}\frac{\delta}{\delta y}\left(M_x \operatorname{sen}\varphi\right) + \frac{\delta^2 M_x}{\delta x^2}\right) +$$
$$+ \left(\frac{N_y}{r_1}\right) + \left(-\frac{1}{r_x}\frac{\delta^2}{\delta y^2}\left(M_y r_x\right) + \frac{1}{r_1}\frac{\delta M_y}{\delta x}\right) + \qquad (7)$$
$$+ \left[\frac{1}{r_x}\frac{\delta M_t}{\delta x}\frac{\delta r_x}{\delta y} + \frac{\delta^2 M_t}{\delta_x \delta_y} + \frac{1}{r_x}\frac{\delta^2}{\delta_x \delta_y}\left(M_t r_x\right)\right]$$

where the sums of the terms enclosed in brackets, functions of the internal actions previously obtained, indicate in the order: the axial strength of the arches, the bending strength of the arches, the axial strength of the cantilevers, the bending strength of the cantilevers, the torsional strength.

Deflections of the crown arch of the Val Gallina dam deduced from the tests on the final model, varying the deformability of the rock foundation as we have said, agree closely with that obtained directly with measurements made on the prototype.

At the end also the model of the Val Gallina dam was subjected to the ultimate load tests by increasing gradually the intensity of the hydrostatic load. The collapse came for a load about ten times the normal one, with interesting flow strains which were registered by means of oscillograph recorder equipment.

b) Together with the above described tests on comparatively slender and thin arch-dams, built across very open U-shaped gorges, the occasion presented itself for approaching the problem—having certain reciprocal aspects—of the arch dams for very high and narrow gorges with almost parallel walls. There were two principal instances to be considered in Italy: that of the Santa Giustina Dam (Edison Group Co.), having a height of nearly 150 m. and a maximum span of about 80 m., and that of the Vajont Dam (former solution), over 200 m. high with a mean span of about one-half the height, but tending to expand towards the top section.

The tests on the model of the Santa Giustina Dam were all carried out before the year 1941 in the Polytechnical School of Milan; their purpose was to compare the results with those of the traditional calculation for elastic arches, as well as—for the lower section—to compare them with those of the previously mentioned photo-elastic tests.

The 1:66 scale model was stressed under the action of pure water load. The weight effect was not brought into play: actually, for well-known reasons, it contributes directly to the stresses in the cantilevers and indirectly also to those in the arches.

The cast was made in plaster of Paris mix; this caused slightly different moduli, higher in the top section and lower in the bottom (thicker) section of the model. This was taken into account in passing from the model results to those inherent to the prototype.

Among the other findings, the values of the main tensile stresses, based on extensometer measurements, and the trend of the isostatic lines were deduced here too. It was noticed that, as a rule, such values differred only a little from those pertaining to the arch-cantilever lattice and this because the

directions of the main tensile stresses nearly coincide with those of the arch-cantilever lattice, with the exception made for the section close to the bottom.

The tests on the Vajont Dam models have been so far developed in three subsequent stages or versions, corresponding to three different models that we shall list them in the order "A," "B" and "C" (Fig. 9). A fourth model, the "D" type corresponding to the new solution studies for the dam, which will attain to 260 m. of height, shall be tested next in the I.S.M.E.S.' tower.

The first two models, tested around 1940, had been made in order to investigate the static behaviour of the lower section of the dam, up to about 100 m. in height, inasmuch as the design was considering comparatively thick arches, which, given the limited span, stood out as extremely massive. In the "A" version it was attempted to keep the abutments as much as possible adhering to the gorge walls; the structure was therefore highly dissymmetrical, with comparatively very thick arches and reduced central angles. In the "B" version it was instead attempted to reduce as much as possible the dissymetry of the structure—at the expense of greater excavation—as well as to reduce the thickness of the arches increasing considerably the central angles and the slenderness compared to the "A" version, with an overall volume reduction of about 30 per cent. As far as the profile of the arches was concerned, the greatest advantage was taken of the photo-elastic tests previously mentioned. The results obtained from the tests carried out on the models, cast in a plaster of Paris-celite mix, emphasized the fact that the static behaviour of the "B" type was on the whole somewhat more favourable than the "A" type (Fig. 10). The dissymetry of the dam considerably affects the trend of the stresses; for these reasons, the "A" version shows much more pronounced inequalities between the stresses at the abutments, and as a consequence, in ultimate analysis, the maximum stresses were found to be lowest in the "B" version, which was more symmetrical than the "A" version. The water load is fundamentally carried by the arches, whereas the stresses along the cantilevers are here comparatively moderate.

The first two models were also tested for the weight effect, due of the upper part—not modelled—of the dam, and these tests showed that such effect was discharged sideways, as though in the framework of the structure some vertical arches had been created to support it; as a consequence in ultimate analysis, in its lower part, at the bottom, the vertical compressive stresses due to the weight was found to be only a small fraction of the very high stresses expected.

The static behaviour of the models was undoubtedly affected by the fact that the upper part was missing; it was therefore considered advisable to design and subsequently test another model of the entire dam. In the designing of such a structure, those variations were taken into consideration which the analysis of the preliminary tests suggested. This model, the "C" version, could only be constructed after the end of the war (1945-47) and was tested merely for the action of the water load (Fig. 11) with the principal purpose of checking whether it was desirable to further limit the thickness of the lower section; to give to the top section a second, limited curvature, and to collect all elements necessary for the final study of the structure, which it is foreseen will reach a height of 250 m. or 840 feet ("D" version).

In order to determine the effects of the local settlements of the material, particularly at the large abutment surfaces, it appeared appropriate to make two complete series of tests and of measurements, the first—known as the "normal load series"—with measurements taken over a load range limited to

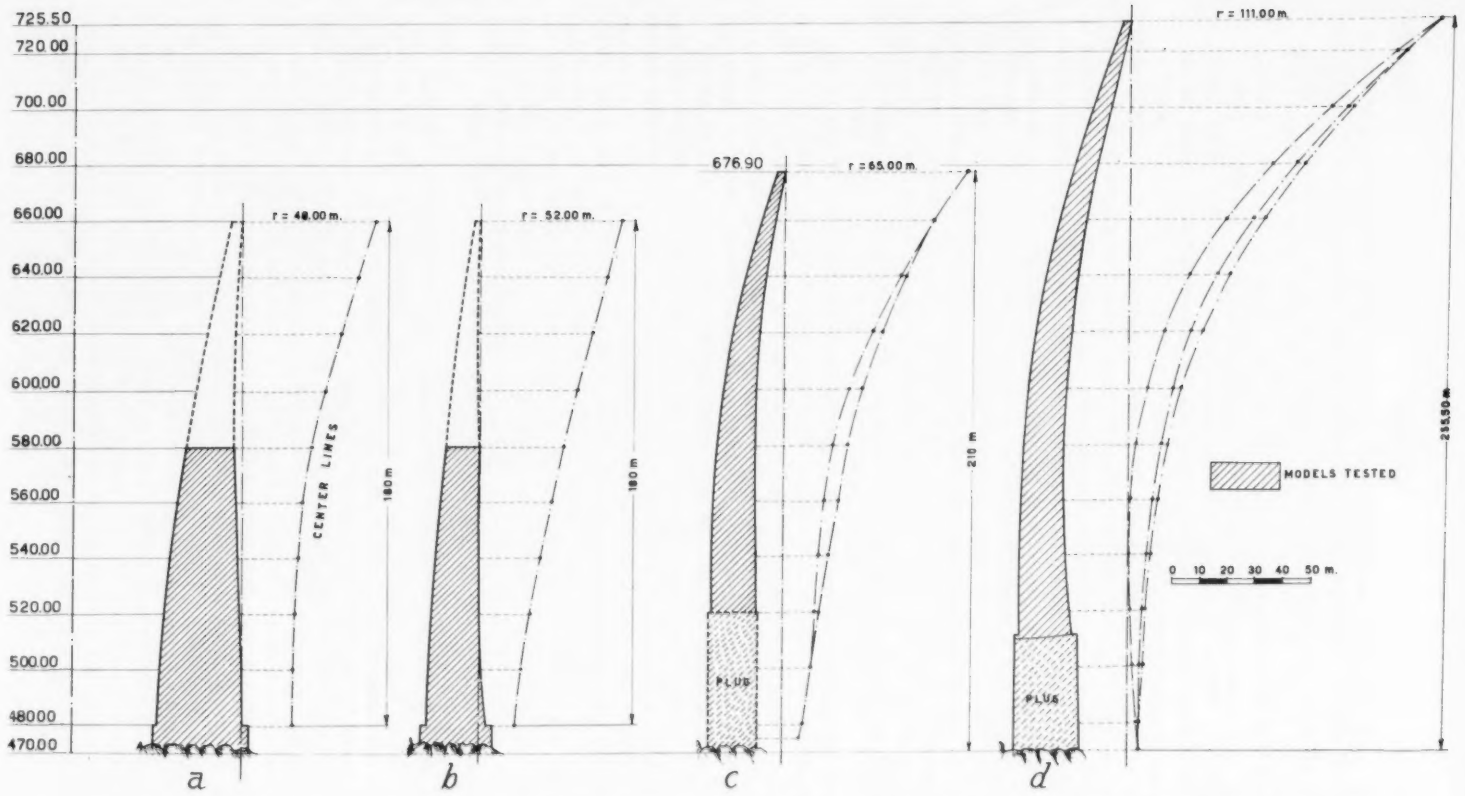

Fig. 9. Vajont Arch Dam: the Cross Section of the Three Models Tested During Preliminary Studies, and that (D) of the New Type to be Tested.

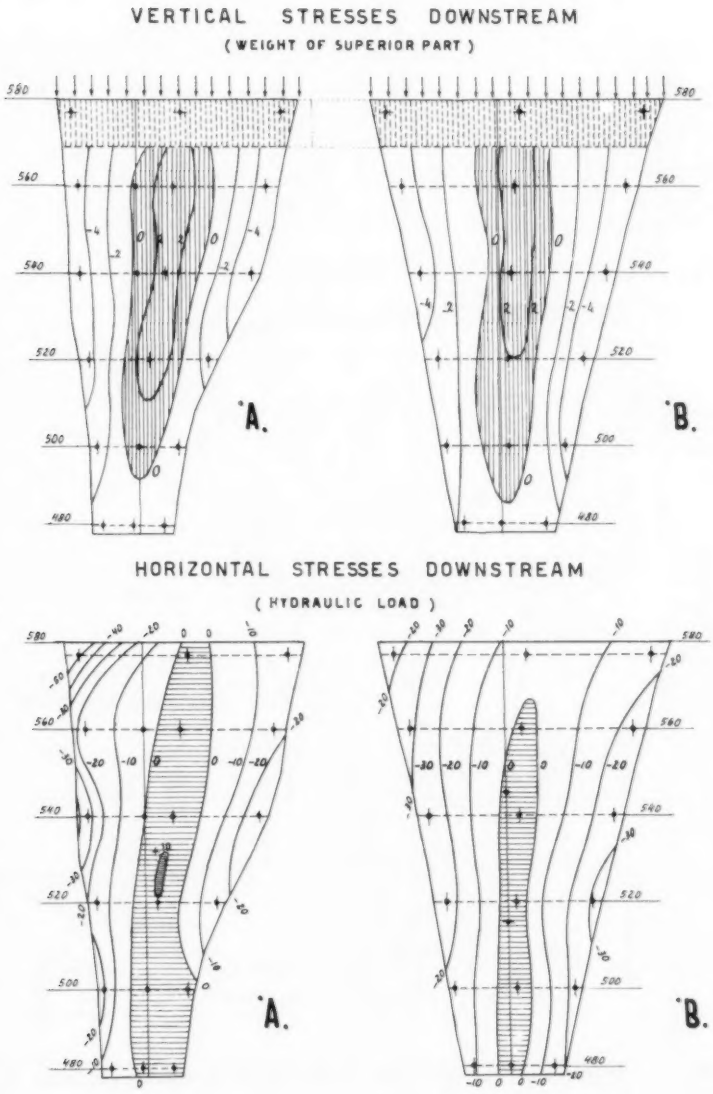

Fig. 10. Results on Comparison Tests Between Types "A" and "B" for the Preliminary Study of the Lower Part of the Vajont Arch Dam.

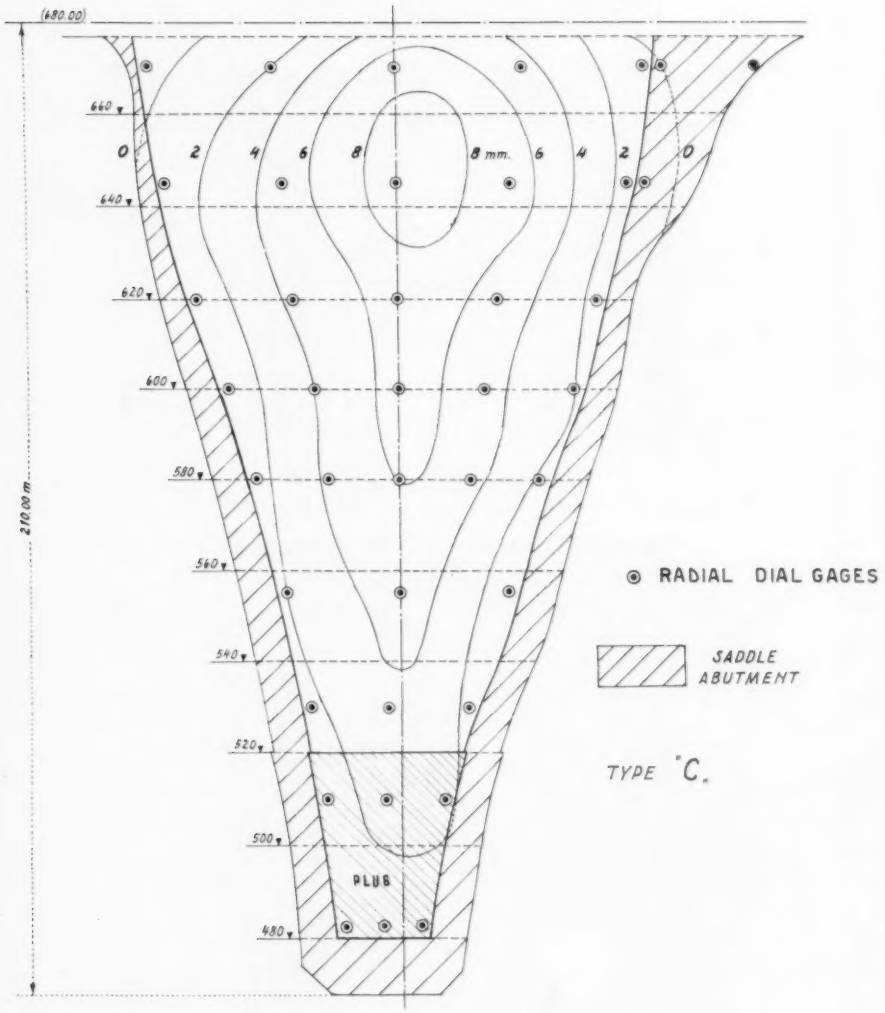

Fig. 11. Curves of Constant Deflection on the Downstream Face of Model "C" of the Vajont Arch Dam.

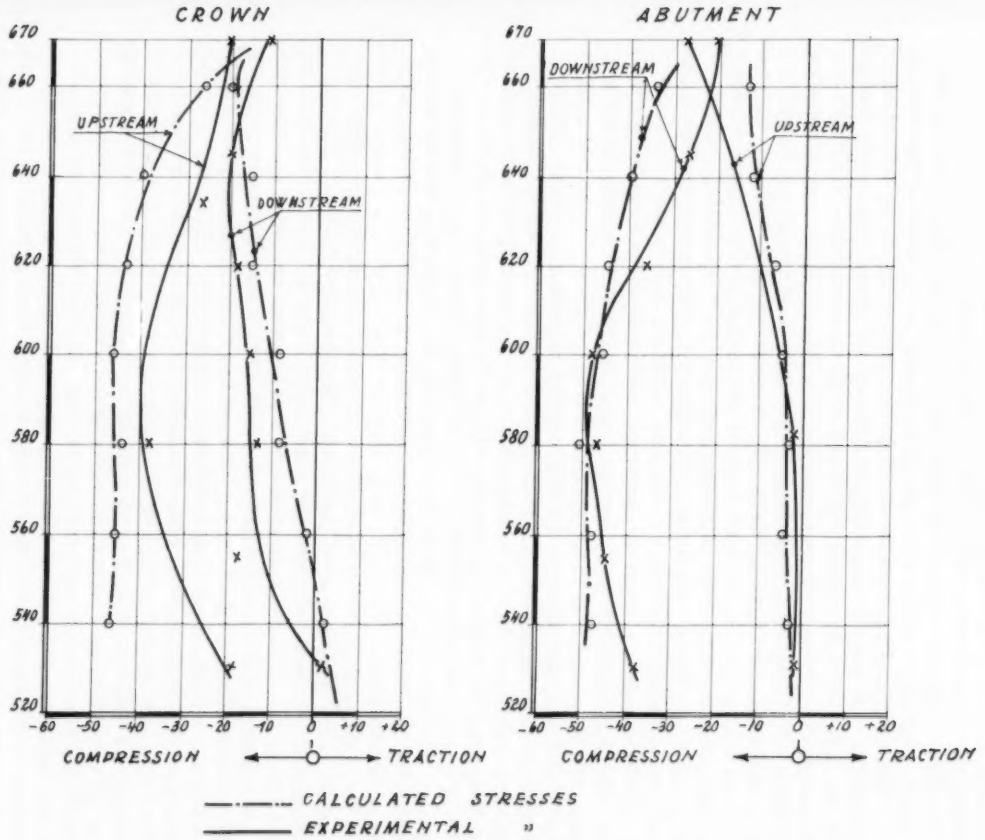

Fig. 12. Comparison Between Calculated and Experimental Stresses in the Arches of the Vajont Dam, Type "C."

a normal water load intensity, p_o; the second, with measurements taken over a load range included between the previous maximum intensity and one twice as big, $2p_o$.

From the results obtained on the model, those corresponding to the prototype (at full reservoir load) were derived in the two cases (Fig. 11); the noncoincidence of the values at corresponding points gave the measure of the settlement effects. It is interesting to note how the stress peaks are reduced in passing from the first to the second series of tests.

These various tests carried out on models of high and narrow dams have brought to light the fact that their static behaviour is distinctly different from that of the structures previously considered: the arch effect is here distinctly prevailing over the cantilever effect, already at normal loads, and, if symmetry is sufficiently conserved, the trend of the isostatic lines follows the arch-cantilever lattice, as it was possible to ascertain experimentally. The agreement with the elastic calculation (Fig. 12) is good enough. Only at the bottom the cantilever effect prevails over the arch effect.

Special mention should be here made to the abutment ties. Clamped setting was obviously done away with, replacing it with a mere leaning of the dam on the rock: about the profile of this bearing, particularly for the deeper arches, an abutment following a trend on a radial surface would have required a considerable amount of excavation and hence large amounts of concrete. Stepped abutments were then designed, which were adopted in the first models without giving rise to difficulties during the tests. In the last model of the Vajont scheme, however, as in the model of the Lumiej Dam, which will be dealt with further on, it was preferred to do away with the steps, adopting a continuous support saddle ("pulvino") and leaning the dam itself up against it along an accessible perimetral joint (protected upstream by a watertight strip, and an asphaltic sheet with filler block); this in order to eliminate the concentration of tension stresses, the steps should have produced and to facilitate the shrinkage and the thermal strains by reducing the consequent stresses.

c) The Lumiej Dam of S.A.D.E., finished in 1947, is an intermediate type in respect to the previous ones, the Lumiej dam having a height of 136 m. and a span at the crown of about the same length, with a V shape rather closed at the bottom.(6) The dam was built with a perimetral abutment joint and with the interposition of a support saddle cast between the dam proper and the rock shoulders containing it. The purpose of the support saddle, beside the practical one already mentioned was that of reducing the values of the normal pressures exerted against the rock, spreading them on a wider surface.

The tests carried out on models (4)—cast on a 1:60 scale, in a plaster celite mortar—had the fundamental purpose to compare the stresses occuring under the normal elastic conditions, with those obtained from calculation; and here the problem was particularly interesting since the designed structure had also a limited second curvature approaching the "dome-arch" type. Moreover, testing was pushed on, through the execution of ultimate load tests, up to the collapse of the model, checking its margins of safety, as had already been done after the Rocchetta Dam, for several other Italian dams.

As far as the dam abutment zone was concerned, the tests did actually indicate that the stresses on the up and downstream faces due to the water load were contained within tolerable limits, but they said nothing about either their local concentrations, close to the abutments, the trend of such enhancements within this zone and in the support saddle, or their increment when thermal action should add to the water load.

On the other hand the calculation, made according to the elastic theory assuming the arches to be clamped into the abutments, was leading—particularly when thermal action were considered in the most unfavourable period (winter)—to pressure curves which showed, near the abutments, eccentricity towards the intrados, such as to cause remarkable tensile stresses along the up stream face. These stresses although tolerable in themselves, were anything but logical, their presupposing a continuity which in effect did not exist. It was therefore to be presumed that in reality, thanks to the perimetral joint, a certain fraction of the abutment zone towards up-stream would remain practically inert—i.e., unstressed—whilst towards down-stream the compressive stresses should increase. Special model testing was organized, limited to those arches of the dam which (from calculation), appeared to be more stressed, such as to furnish quantitative elements useful in clarifying the static behaviour of the abutments zone and of the support saddle. Two successive series of tests were thus carried out on two-dimensional models: the first, in 1944, on celluloid models; the second, the following year, on large concrete 1:10 scale models.

As it was foreseen, the tests showed that the stress values, here really localised, were considerably affected by the condition of the bearing surfaces. It appeared therefore advisable to extend the tests and obtain results both in the case of surface cast directly one against the other, and in the case of a surface brought into contact with the interposition of a thin coat of cement mortar.

It was ascertained that the value of the local compressive stresses at the corners (edges) of the down stream face increased, reaching at the arch abutment as much as 80 Kgs. per sq. cm. (arch at 900 m. elevation) against the 60 predicted. It is not surprising to have found here higher values than in the previous tests and in the calculation, since only here strongly localized stress values had really been obtained.

Among the arch dam models recently tested at the I.S.M.E.S. Institute, the author wishes to further mention those of the Rio Freddo Dam (C.I.E.L.I.) and the Pontesei Dam (S.A.D.E.) the former quite interesting because of the abutting of the vault, which is not made directly on to the rock, but onto gravity buttresses; the latter, because it was designed as a rather thin dome type, with remarkable compressive stresses and gave very good results. This model moreover was utilized for an experimental study which, even though not concerning the prototype itself, supplied the solution of not too easy a problem (raised in connection with other Italian dams) concerning the possibility of damming a large U gorge formed by two high strength rocky side walls and a bottom foundation that did not give sufficient guarantees of resistance under vertical loads. A small cleft had been cut horizontally in the bottom section of the model, through from upstream to downstream face. It was then investigated what strength resources the dome structure possessed to discharge sideways also the load of its own weight, should the bearing effect fail at the bottom. This was achieved by measuring the stresses induced on the model by alternate cycles (loading and unloading) of the self-weight equipment.

In concluding this chapter, the author wishes to point out the high values of the "safety coefficient" (whose significance has already been discussed) from 8 to 12 shown by testing the models of this type of thin arched dam. The reason of this is to be sought in those peculiar strength resources which are a property of the three-dimensional vaults and dome dams, resources which,

difficult to frame in a schematical calculation, are instead perfectly reproduced by the model.

Three Dimensional Models of Arch-Gravity Dams

Designs and construction of arch-gravity dams have been developed in Italy since some years ago, which are particularly suitable for the closure of a gorge having high span-height ratio, when the condition of the foundation rock allows their employ.

The first example of a complete design study for a large structure of such a type was the dam built by S.A.D.E.[6] across the Piave River, which, finished in 1949, stands out to this date as an outstanding example of its kind. The tests made on models of this dam turned out to be of the greatest importance, both for the evaluation of its safety margin and as an aid to the theoretical studies made concurrently with the tests, with a real team-work collaboration. (Various procedures were followed, including the "Trial Load Method" and the extension of the Tolke calculation.)

The Pieve di Cadore Dam has a maximum height, at the narrow gorge where the river flows, of 112 m., a crown span of about 330 m. and very large central angles. The section of the structure actually resisting as an arch-gravity type, of a mean height of 55 m., has an almost symmetrical development in respect to a middle vertical plane and is for the greatest part placed on the rock (limestone) and for the remaining part on a plug closing the deeper and narrower part of the gorge. The plug was designed almost exclusively with the aid of models testing which permitted the evaluation of its behaviour under water load.

Such tests were carried out following two procedures which supplemented one another. According to the first, tests were made using models of considerable dimensions (1:25 scale) of suitably compounded concrete, capable of reproducing both the elastic and the elasto-plastic static behaviour; according to the second procedure, the study of the elastic behaviour was improved using small photoelastic models. The first investigation, the closest to reality, reaches and evaluates the unelastic behaviour of the material and deduces the actual safety coefficient of the plug; but, unlike the second, only permits a limited number of measurements and is experimentally less exact. The second, instead, repeatable at will, lends itself to the comparative study of different profiles, successively corrected, with particular reference to the abutment zone.[10]

The most important tests were however those connected with the study of the whole dam in its curved plate continuity, bringing into play the non-computable and yet important factors, such as the asymmetry of the dam caused by the existence of the plug, the presence of the perimetral joint and the radial joints, and by the irregular outline of the foundation and abutment sections.

These tests were carried out at Bergamo in a big reinforced concrete double deck basin, on two large models, both in a 1:40 scale.[10] The first of these models was set up in 1947 and was useful, among other purpose, for indicating those variations of design which were later reproduced in the second model, so that this latter (having also been modified in the elastic behaviour of the foundation model according to the latest results of the geological investigation) did faithfully correspond to the final prototype; the results hereinafter briefly mentioned, refer only to this second model.

Two principal tests cycles were made; one at normal regimen of the water load (corresponding to 140 atm. at the jacks), the other at a pressure increased by 50%. A comparison of the results clearly showed the good proportionality of the two series of tests. It may be noticed that due to the high deformability of the rock foundation the displacements at the foot are considerable, particularly in the central zone and that they particularly affect the trend of the total deformations.

The experimental results so far mentioned, obtained directly from the model measurements, offered a first fundamental criterion for judging the static efficiency of the dam. It seemed however convenient to work them out again so to obtain a deeper insight of the structure's strength to compare them with those established by the various methods of calculations, and draw some general conclusions, as far as possible, for structures such as dams of similar type; and since this structure—like other arch-gravity dams later designed and built in these last years in Italy, i.e., Pian Telessio dam—exhibits an ample central zone of nearly uniform characteristics, it appeared convenient to limit the theoretical investigation to this central zone and above to the perimetral joint, the remaining part of the dam being so complicated by local conditions as to frustrate any attempt to make a generalized investigation. For this latter part (in each single case) the model will be the only necessary and sufficient means of solving the problem; it clearly reveals the forming of inclined arches (plongeants) whose static function prevails over that of the vertical cantilevers and horizontal arches. As far as these lateral zones are concerned, therefore, even the usual calculation schemes are but illusory.

In the Piave dam the above-mentioned central zone extends approximately over half of the dam; here the stress trajectories appear as a network with a rectangular mesh lattice; at each elevation, the values of both vertical and horizontal stresses are distributed with fair uniformity on both faces.

In our case, for the evaluation of the water load distribution among the various strength capacities of the dam, equation (7) may be further simplified; in fact:

a) for the whole of the width of the vault here considered errors, we have practically $\tau = 0$; hence $M_{tx} = 0$, which means that for the central zone of the dam the torsional contribution is negligible;

b) it is also noted that $M_y(x) = \text{constant}$; from the expression of the bending strength of the cantilevers, the term $\frac{I}{rI} \frac{\delta M_y}{\delta x}$

c) the term $\frac{N_y}{r_I}$ deserves a special mention.

In the case of a thin shell, it represents the axial strength of the cantilevers, i.e., the contribution of the second curvature. The model results show that in our case N_y is very small; it is however uncertain whether its existence might be attributed either to a real second curvature effect or to an enhancement of the up-stream stress owing to a non-perfect linearity, or still to the existence of a system of subvertical arches within the dam thickness, or finally and more probably, to the contemporaneous presence of some of these factors. In any case, the action is small, probably of the same order of magnitude as inherent errors of the computation procedure. It is therefore practically admissible to neglect the second curvature effects.

From the computations done, the load appears supported almost exclusively

by the axial strength of the arches and the bending strength of the cantilevers. The bending strength of the arches is very small everywhere, with a slight tendency to increase towards the foot of the dam.

Likewise very small, just perceptible at the foot of the dam, as was to be expected from the considerable width of the zone under consideration, was the residual contribution (necessary for the equilibrium equation 1 to be verified), due to the torsional stresses not directly deduced from model measurements.

A direct comparison between the results obtained from the tests and those from the calculations is also interesting. Among the latter, we found those of the Tölke method to have a particular significance for all the successive stages of the structure design, up to the final one, as tested on tridimensional models.

From the brief discussion which follows it will appear why it is justified to adopt such a method of calculation as fundamental for these types of very large dams—having large central angles and vertical radial section of nearly uniform characteristics for a considerable width.

A first comparison, always limited to the central zone, may be made between the load distributions between arches and cantilevers, and the consequent mean axial forces in the arches as well as the stresses at the up and down-stream face of the cantilever. The agreement is, on the whole, good and would further improve if a modulus ratio of 1/7 between rock of the foundation and concrete of the dam, had been assumed for the calculation, as the one obtained in the tests, which would only lead to a slight decrease of the contribution of the arches.

Now, the most notable difference between the Tölke method and the experimental one lies in the fact that the former does not take into consideration the effect of the side abutments and considers the dam as a sector of a shell. The good agreement among the stresses shows that, in the central zone at least, the arches having central angles around 130°, therefore much smaller than 360°, little affects the values of stresses.

The agreement is also good in the practical centering of the pressure curve in the arches. This may be explained considering that, whilst the abutment tie tends to create bending stresses, the considerable rigidity of the cantilevers tends to annul them quickly.

A second comparison may be made between radial (horizontal) deflections; in fact, they constitute an integral phenomenon, responding to the strains acting both in the central and the abutment zones, and are therefore apt to supply indications, even though of a general character, on the behaviour of the structure as a whole. Now, it was observed[10] that the test deflections are intermediate between those obtained from the Tölke calculation and those that would be obtained considering the arches as independent, clamped into the abutment and loaded with the Tölke portion of hydrostatic load p_a. The explanations of the phenomenon may be sought in the fact that the load is to be considered as increasing from the key towards the abutments (and it certainly is such at least close to the abutments), and in the limited extension, ascertained above, of the bending stresses due to the abutment tie. Both these causes, in fact, are concurrent in reducing the central line deflections.

In conclusion, the Tölke method seems to be fairly well suited to outline the behaviour of a dam having a high span-height ratio, uniform vertical sections and curvatures (for a sufficiently wide angular range).

If the Piave arch gravity dam shows a special interest besides the one it raises on account of its priority in order of time, various other models,

among the many of the arch-gravity dams tested at the I.S.M.E.S., would also deserve a special mention on account of the peculiarity of their problems and in order to emphasize the substantial contribution brought by their testing on models to the solution of such problems. In the following pages we shall briefly discuss the testing of the three other models of Italian important arch-gravity dams.

b) The Beauregard Dam (S.I.P. Turin) has a maximum height of 135 m., a development at the crown of about 400 m. and the thickness increasing along the main section from 5 m. up to a maximum of 45 m. at the foot. The curvature radius is of 163 m. and the central angle 132° at the crown.

The main peculiarity of the problem of this large dam is the considerable dissymmetry of the elastic characteristics of the rock foundation, the ratio between the moduli of the two opposite abutment banks being 1:10; a further complication rises then from the presence of a huge pocket of "mylonitic" material at the foot of the yielding abutment (left side), of such proportions as to require the construction of a substantial concrete masonry sub-foundation.

For the designing of this dam, the contribution of model testing was remarkable ever since the preliminary studying stage. The task of checking the validity of the theoretical calculation which were to be applied for the solution of the plain problem of an isolated thick arch with dissymmetrically yielding abutments, was entrusted to the ISMES. We have solved this problem by creating special testing equipment (fig. 7) with which it was possible to test on plane models several arches measuring the magnitude of the pressure curve displacements in respect to the central line; such displacements were considerably close to the stiffer abutment, tending to be very little towards the more yielding abutment and to be maximum, towards the extrados, in a section that no longer was the key one (as the case would be with a symmetry of abutment conditions), but was shifted in respect of it towards the more yielding abutment. Based on these preliminary results we also made the suggestion to draw the arches (horizontal sections) of the dam, not according to a symmetrical profile, but adopting instead a greater curvature of the axis on the side of the more rigid foundation.

Starting from these presuppositions, a first complete design was worked out and a 1:50 scale model was constructed according to it, faithfully reproducing the elastic characteristics of the foundation, including the reconstructed pocket. This model, cast in a special pumice mortar concrete with an "effectiveness ratio" $\zeta = 4$ was tested in the ISMES big testing tower. From the results obtained, the possibility arose to proceed to a rational reduction of the structure volume, thus achieving the double purpose of increasing the structure efficiency and reducing at the same time considerably its cost.

The same model was consequently chiselled out in order to make the variation agreed upon, and was subsequently subjected to a new series of tests.

Finally, the "ultimate load tests" were executed with very good results. The first cracks (downstream) appeared after having increased the dead load twice (maximum of the experimental equipment) for a hydrostatic load 5 times the normal one. Successively, this load was yet augmented and for an intensity of 6.75 times the normal value the model failed through yielding of the more deformable abutment.

The Pian Telessio Dam (A.E.T. Turin) introduces into the field of the "span height" ratio values, usually pertaining to the arch-gravity dams, a peculiar

value due to the exceptional development of the crown arch, which is equal to about 6.5 times the maximum height. This latter is in fact 80 m., while the crown has a mean development of 510 m. (1,700 feet), the maximum value till now reached for the arch dams.

A preliminary designing study having been executed, ISMES was requested to proceed with the investigation of a model also cast in pumice concrete on a 1:70 scale; the mountain was faithfully reproduced using deformable materials having about the same moduli as those of the dam, ($E_c/E_r = 1$), resulting from tests made "in situ." The model was subsequently tested for the full reservoir water load condition, after having been subjected to the equivalent load of its real dead weight applied at open joints.

The Cancano Dam (A.E.M. of Milan) has required for its designing the overcoming of substantial difficulties. The first of such difficulties is represented by the necessity of building the dam in two subsequent stages. The first stage construction reaches a height of 136 m. with a span of 330 m. and shall have to be placed under load for a period today not yet determined; the second stage construction shall have to be later cast on the first and will bring the height of the dam to a maximum of 176 m. (575 feet), with a crown span of 460 m. (1,600 feet).

The foundation rock (limestone) which shows a pronounced anisotropy ($E_{max}/E_{min} = 4$) due to the inclination of the strata outcrops on the sides of the valley, so that the flow of the stresses transmitted by the saddle of the dam to the rock is perpendicular to the bank in one side and parallel to it in the opposite one. The model, constructed on a 1:50 scale, was so far limited to the first stage construction and faithfully reproduces the anisotropy characteristics of the rock, this effect being obtained by the interposition of rubber sheets conveniently perforated so as to permit their expansion in a lateral direction.

This experimental study on model disclosed how a rational dimensioning of the structure may lead to a substantial reduction of the dissymmetry effects mentioned above.

The "ultimate load tests" also gave very good results giving a factor of safety for the structure of about 6.5.

c) It may be interesting to note how the elastic results obtained in testing these and other arch-gravity dams have emphasized some systematic differences with the theoretical calculation. The author believes that it may be partly due to the fact that the theory follows these principal postulates:

1. Linearity between stresses and strains (Hookes' law);
2. Structure continuity (absence of the radial and perimetral joints);
3. Symmetry of the dam and mountain foundations in respect to a main vertical section (generally);
4. Settlements of the foundation obtained by extending the (Vogt, Boussinesq) theory on the deflections of a semi-space due to a uniform load on a rectangular abutment area;
5. Law of conservation of the linear diagram in the stress distribution through the thickness (both for arches and cantilevers).

Based on the above postulates, we have compared the values of the stresses obtained from calculation (with $E_c/E_r = 2$) for the central zone of the Cancano Dam with the corresponding stresses deduced from the model, which are affected by the ascertained space dissymmetry of the structure. The same comparison was made for the other two dams already mentioned.

The following findings were clearly arrived at with the tests:

1. Tensile stresses on the downstream face in a vertical direction (cantilevers) somewhat greater than those indicated by the calculation;
2. Compressive stresses reduced on the downstream face and increased on the upstream one (therefore beneficial to stability) in the middle and bottom part of the cantilevers;
3. Higher stresses in a horizontal direction; i.e., higher arch contribution to the overall strength.

To better support these findings, which we deem to be of a general character (for arch gravity dams of the same type), we have thought it interesting to further elaborate the results obtained.

By assimilating the dam to a shell, as we have already said, from the strain measurements read directly on the model, by calculation we have traced back to the evaluation of the (water) load distribution among the main strength elements of the space structure, applying the relation (7).

The arguments the writer has already had occasion to assert, in connection with the study of the static behaviour of arch-gravity dams, appear evident. It also appears quite evident that to the fundamental strength elements, i.e., arches and cantilevers, a not negligible additional effect is to be added in the lower part of the structure due to complementary strength participation (torsional strength) which, in the ultimate analysis, do mitigate the load share supported by the traditional arch-cantilever lattice.

SUMMARY AND CONCLUSIONS

In conclusion it will be seen that the experimental researches on models have given, in Italy, a large contribution for the rational design of structures and, particularly, of the arch-dams.

After having summarized the actual possibilities of structural models, obtained from the theory of similitude and from laboratory researches, with particular reference to the researches of the ISMES works Institute the paper gives a description of the most important cases of Italian arch dam studied, till now, by means of models and reports some of the principal results obtained.

The experience and knowledge of the authors of the project for a dam permit, during the preliminary studies (in according with the topographic and geologic conditions) to establish a first economical balance among the different kinds of dams which may be considered as convenient to construct, and analytical calculations may be carried out for providing a first idea of the elastic stresses to be present in the dam.

But the possibility of rational economies in the design of the dam, of saving concrete, the confirmation of the principal stresses foreseen and determinations of the true factor of safety are most reliably accomplished by the aid of model-tests which, in several cases of Italian dams, have finally decided between the different types of dam-projects initially proposed.

Moreover when the arch dam exhibits dissymmetry, heterogeneous foundations, irregular abutment-lines and other irregularities; such as joints, spillway opening, interior tunnels and so on, only the model-tests can give good results. Such results have been recently confirmed by their good agreement with the measurements made on constructed dams.

BIBLIOGRAPHY

1. G. Oberti, B. Bonfioli: Research on the stresses of arch dam rings Second Congress on large dams Washington, D. C. 1936.

2. Bureau of Reclamation: Trial load method of analysing arch dams Bulletin I, Denver 1938.

3. G. Oberti: Risultati di studi sperimentali eseguiti sopra un modello di diga - "L'Energia Elettrica" 1940.

4. G. Oberti: La diga del Lumiej - Criteri di progetto e studi sperimentali - L'Energia Elettrica" 1948.

5. A. Berio: Alcuni risultati sperimentali ricavati da un modello di diga ad arco - "L'Energia Elettrica" 1953.

6. C. Semenza: The most recent dams by the Societa Adriatica di Elettricita (S.A.D.E.) in the eastern alps. "Civil Engineers" - September 1952.

7. C. Semenza: L'impianto idroelettrico Piave-Boite-Mae-Vajont- "L'Energia Elettrica" febbraio 1955.

8. G. Oberti: Diga di Val Gallina - Criteri di progetto e richerche sperimentali - "Serie" 1955.

9. G. Oberti: L'Etude rationelle et économique des grands barrages en béton par l'utilisation des essais sur modèles - Cinquième Congrès des grands barrages - Paris, 1955.

10. A. Danusso, G. Oberti: Studi teorici e sperimentali sulla diga arco-gravita del Piave - "L'Energia Elettrica" 1955.

11. E. Fumagalli: Communication sur les matériqux pour modèles statiques de barrages en béton - Cinquième Congrès des G. B., Paris 1955.

12. E. Lauletta: Communication sur deux appareils de mésure pour modèles statiques de barrages - Cinquième Congrès des G. B., Paris 1955.

DIVISION ACTIVITIES

POWER DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

August, 1957

Mr. John F. Bonner, M. ASCE, Assistant to the Vice President and Chief Engineer, Pacific Gas & Electric Company, has been appointed by the Board of Direction as a Member of the Executive Committee of the Power Division for a term expiring October 1961. Mr. Bonner will succeed Mr. Henry Lutge whose term of office expires in October 1957.

October 1957 Convention Program

Monday, Oct. 14	R. A. Sutherland presiding.
Afternoon	Civil Engineering features of the Linden Plant of New Jersey Public Service Company, A. Verduin, Public Service Electric & Gas Co., New Jersey.
	Circulating Water Systems of Steam Power Plants R. T. Richards, Ebasco Services, Inc.
	HAAS Underground Power Plant of the PGE Co., J. B. Cooke, PG&E Co.
Tuesday, Oct. 15	R. A. Sutherland presiding.
Morning	Underground Power Plants in Italy, Engineer of S.A.D.E. Co.
	Underground Power Plants, A. Ackerman, Cons. Engr.
Tuesday, Oct. 15	G. H. Von Gunten presiding.
Afternoon	Underground Power Plants in Yugoslavia V. Yevdyevich, Dir. Inst. of Hydr. Eng. Belgrade
	Underground Plants in Sweden, K. V. T. Nilsson, Swedish State Power Board
Wednesday, Oct. 16	Visit to Linden Plant of the New Jersey Public Service
Afternoon	Company. The trip by bus will take about 45 minutes each way, allowing one hour for inspection of plant. This plant supplies some steam and power for the Esso Bayway Refinery and in return receives residue oils as fuel.
Thursday, Oct. 17	A. Ackerman presiding.
Morning	Underground Power Plants in Switzerland Dr. A. Kaech, Cons. Engr. Luzerne
	Underground Power Plants in Canada A. W. F. McQueen, H. G. Acres & Co.

Note: No. 1957-15 is part of the copyrighted Journal of the Power Division of the American Society of Civil Engineers, Vol. 83, PO 4, August, 1957.

Thursday, Oct. 17 G. J. Vencill presiding.
Afternoon Ambuklao Underground Power Station. A. Eberhardt, Harza Eng. Co.
The Sudagai Underground Power Plant, Japan
T. Mizukoshi, Tokyo Electric Power Co.
Further discussion of Underground Power Plants.

Future Programs

The Chicago Convention in February 1958 will include papers on the Dresden Nuclear Power Station with an inspection trip to the site planned. There is still space on the program for additional papers and Members of the Power Division are invited to submit papers for consideration.

The June meeting in Portland will include a symposium on rock fill dams.

The program for the October 1958 Convention will consist of a symposium on "Civil Engineering Aspects of Thermal Plants" and a very complete coverage of this subject is planned.

Committee Meetings

Three Power Division committees held meetings at the Buffalo Convention last June and report very profitable meetings. The Committee on Nuclear Energy discussed the subjects "Choosing a Site For a Nuclear Power Plant" and "Cooperation Between Manufacturer's Development Engineers and Civil Engineers." There was also a general review of the subject "Training and Education of Engineers for Nuclear Power." The other committees meeting were the Committee on Progress in Power Plant Design and the Committee on Operation and Maintenance of Hydroelectric Stations.

Board Committee Commends Power Division

A letter from the Executive Secretary quotes from a report to the Board of Direction.

> 'The Waterways and Harbors Division, and the Power Division continue to do the most active publishing of Technical Divisions in point of members enrolled. Between February 1956 and January 1957-. . . (the) Power Division, with an enrollment of less than 8% of total, presented almost 14% of all papers in technical journals in that period of one full year. . . .'

The Board directed that the Executive Secretary send letters of commendation to the Power Division for effective development and processing papers for publication. Much credit for this fine work is due the members of our Division Publications Committee and to Mr. Sutherland for backing the Symposium on Arch Dams which resulted in a large number of papers for publication.

World Power Conference

The Power Division has been invited to participate in the World Power Conference to be held in Montreal, September 7 - 11, 1958. We have agreed to contribute a paper on the subject of economic trends of production in hydraulic energy, and arrangements are being made for the authorship of this paper. Other papers to be presented in this category are "Modern Hydroelectric Potential Evaluation" by E. G. Nielson, Assistant Commissioner of the Bureau

of Reclamation and "Hell's Canyon Hydro Development" by T. E. Roach, President, Idaho Power Company.

Suggestions Welcome

All ASCE Members are invited to participate in the work of the Power Division by committee activities and preparation of papers. Suggestions are always welcomed by the Executive Committee and any Member is invited to participate in our work.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1113 is identified as 1113 (HY6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1956.

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AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048 (SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068 (WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092 (HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103 (IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c]; 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127 (SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

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JANUARY: 1136(CP1), 1137(CP1), 1138(EM1), 1139(EM1), 1140(EM1), 1141(EM1), 1142(SM1), 1143(SM1), 1144(SM1), 1145(SM1), 1146(ST1), 1147(ST1), 1148(ST1), 1149(ST1), 1150(ST1), 1151(ST1), 1152(CP1)[c], 1153(HW1), 1154(EM1)[c], 1155(SM1)[c], 1156(ST1)[c], 1157(EM1), 1158 (EM1), 1159(SM1), 1160(SM1), 1161(SM1).

FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)[c], 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238(WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246 (HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253(IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c],1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303 (ST4), 1304(ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318 (ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338(SA4), 1339(CO1) 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c], 1350(PO4), 1351(PO4).

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c. Discussion of several papers, grouped by Divisions.

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